EARLY RETAINING WALLS (GN1) WALL 09.05R-B FDC-054

I-405; RENTON TO BELLEVUE WIDENING AND EXPRESS TOLL LANES PROJECT

Design Calculations:

Soil Nail Wall Facing

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Wood Environment & Infrastructure, Inc.
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Introduction

Soil Nails designed by geotechnical designer, and design information provided for the design of the Wall Facing.

References

1 WSDOT Bridge Design Manual M 23-50.18, June 2018 (BDM)
2 WSDOT Design Manual M 22-01.16, February 2019 (DM)
3 FHWA NHI 14-007, Geotechnical Engineering Circular 007, Soil Nail Walls Reference Manual (2015)
4 FHWA0-IF-03-017 Geotechnical Engineering Circular No. 7, Soil Nail Walls (2003)
5 Geotechnical Design Information - Attached
6 WSDOT BDM Plans for Soil Nail Walls
7 AASHTO LRFD Bridge Design Specifications

RFP Section 2.13.4.3 Retaining Wall Design Criteria

- **Soil nail walls in accordance with Section 6-15 of the Standard Specifications.**

Fall protection shall be provided at the top of all retaining walls and retaining wall terraces in accordance with Section 730.04(7)(b) of the WSDOT *Design Manual*. Fall protection shall be of the Standard Guardrail type, as described in WAC 296-155. Chain link fence is acceptable for fall protection, provided it includes a top rail and middle rail with at least 1.5 inches nominal outside diameter. Timber shall not be used as a material type for Standard Guardrail. For fall protection features that are exposed to the public, design of railings shall be in accordance with Chapter 13 of the AASHTO *LRFD Bridge Design Specifications*.

WSDOT BDM, 8.1.2

E. Soil Nail Walls

The basic concept of soil nailing is to reinforce and strengthen the existing ground by installing steel bars called "nails" into a slope or excavation as construction proceeds from the "top down". Soil nailing is a technique used to stabilize moving earth, such as a landslide, or as temporary shoring. Soil anchors are used along with the strength of the soil to provide stability. The Geotechnical Engineer designs the soil nail system whereas the Bridge and Structures Office designs the wall fascia. Presently, the FHWA Publication FHWA-NHI-14-007 "*Geotechnical Engineering Circular No. 7 Soil Nail Walls*" is being used for structural design of the fascia. See Bridge Standard Drawing 8.1-A4 for typical soil nail wall details.

WSDOT BDM, 8.1.8

8.1.8 Design of Soil Nail Walls

Soil nail walls shall be designed in accordance with the FHWA Publication FHWA-NHI-14-007 "*Geotechnical Engineering Circular No. 7 Soil Nail Walls*" February 2015. The seismic design parameters shall be determined in accordance with the most current edition of the AASHTO *Guide Specifications for LRFD Seismic Bridge Design* (SEISMIC). Typical soil nail wall details are provided in Appendix 8.1.

WSDOT BDM, 8.1.8

1. Expansion Joints

For cast-in-place construction, a minimum of ½ inch premolded filler should be specified in the expansion joints.

WSDOT BDM,Appendix 8.1

Wall Types		Design Specifications
Soil Nail Walls	General	All soil nail walls and their components shall be designed using the publication "Geotechnical Engineering Circular No. 7" FHWA-NHI-14-007. The Geotechnical Engineer completes the internal design of the soil nail wall and provides recommendations for nail layout. The structural designer will layout the nail pattern. The geotechnical engineer will review the nail layout to insure compliance with the Geotechnical recommendations. The structural designer shall design the temporary shotcrete facing as well as the permanent structural facing, including the bearing plates, and shear studs. The upper cantilever of the facing that is located above the top row of nails shall be designed in accordance with current editions, including current interims, of the following documents; AASHTO LRFD Bridge Design Specifications, WSDOT GDM and WSDOT BDM.
	Seismic	AASHTO Guide Specifications for LRFD Seismic Bridge Design using the USGS 2014 Seismic Hazard Maps with Seven Percent Probability of Exceedance in 75 yrs (1000 yr Return Period) and the site coefficients or site-specific procedure provided in the WSDOT Design Memorandum dated January 8, 2017 and WSDOT BDM Chapter 4.
	Traffic Barrier	Moment slab barrier shall be designed in accordance with the WSDOT BDM and the AASHTO *LRFD Bridge Design Specifications* Section A13.3 for Concrete Railings considering a minimum TL-4 impact load

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WSDOT BDM, Chapter 8, Section 8.1.9.C Fall Protection

C. Fall Protection

For retaining walls with exposed wall heights of 4 feet or more, fall protection shall be provided in accordance with WAC 296-155-24615(2) and WAC 296-155-24609 and as described in the *Design Manual* Chapter 730.

Fall protection shall be required regardless of the location of a traffic barrier placed behind the wall, unless the traffic barrier has a minimum height of 3'-6" and is either a moment slab traffic barrier located on top of the wall or a traffic barrier constructed integral with the top of the wall.

WSDOT DM 730.04 Design Requirements

(b) Worker Fall Protection

Department of Labor and Industries regulations require that, when employees are exposed to the possibility of falling from a location 4 feet or more above the roadway (or other lower area), the employer is to ensure fall restraint or fall arrest systems are provided, installed, and implemented.

Design fall protection in accordance with WAC 296-155-24609 for walls that create a potential for a fall of 4 feet or more. During construction or other temporary or emergency condition, fall protection will follow WAC 296-155. Any need for maintenance of the wall's surface or the area at the top can expose employees to a possible fall. If the area at the top will be open to the public, see Chapter 1510, Pedestrian Facilities.

For maintenance of a tall wall's surface (10 feet or more), consider harness tie-offs if other protective means are not provided.

For maintenance of the area at the top of a tall wall, a fall restraint system is required when all of the following conditions will exist:

- A possible fall will be of 4 feet or more.
- Periodic maintenance will be performed on the area at the top.
- The area at the top is not open to the public.

Recommended fall restraint systems are:

- Wire rope railing with top and intermediate rails of ½-inch-diameter steel wire rope.
- Steel pipe railing with 1½-inch nominal outside diameter pipe as posts and top and intermediate rails.
- Concrete as an extension of the height of the retaining wall.

A fall restraint system is to be 42 inches high, plus or minus 3 inches, measured from the top of the finished grade, and capable of withstanding a 200 lb force from any direction, at the top, with minimal deflection. An intermediate cable or rail shall be halfway between the top rail and the platform. A toe board with a minimum height of 4 inches will be provided. Post spacing is no more than 8 feet on centers. (See the *Construction Manual* and WAC 296-155 for fall arrest and protection information.) For wire rope railing, the top railing shall be flagged at not more than 6-foot intervals with high-visibility material.

The designer is to contact maintenance personnel regarding fall protection and debris removal considerations.

Contact the HQ Bridge and Structures Office for design details for any retrofit to an existing retaining wall and for any attachments to a new retaining wall.

WAC 286-155-24615(2)

Information	Summary
2) Guardrail specifications. (a) A standard guardrail system must consist of top rail, intermediate rail, and posts, and must have a vertical height of 39 to 45 inches from upper surface of top rail to floor, platform, runway, or ramp level. When conditions warrant, the height of the top edge may exceed the 45 inch height, provided the guardrail system meets all other criteria of this subsection. The intermediate rail must be halfway between the top rail and the floor, platform, runway, or ramp. The ends of the rails must not overhang the terminal posts except where such overhang does not constitute a projection hazard.	Required vert H = 39-45 inches. Design for H = 42 inches
(b) Minimum requirements for standard guardrail systems under various types of construction are specified in the following items: (ii) For pipe railings, posts and top and intermediate railings must be at least 1 1/2 inches nominal OD diameter with posts spaced not more than 8 feet on centers. Other configurations may be used for the top rail when the configuration meets the requirements of (b)(vii) of this subsection.	Using pipe railing option. Must have top and intermediate rails at min 1.5 inch dia.. Vert Posts spaced no greater than 8 ft.
(v) The anchoring of posts and framing of members for railings of all types must be of such construction that the completed structure must be capable of withstanding a load of at least 200 pounds applied in any direction at any point on the top rail. The top rail must be between 39 and 45 inches in height at all points when this force is applied.	Design check using 200 lbs and to review AASHTO loading per RFP
(c) Toe board specifications. (i) A standard toe board must be a minimum of 4 inches nominal in vertical height from its top edge to the level of the floor, platform, runway, or ramp. It must be securely fastened in place with not more than one-quarter inch clearance above floor level. It may be made of any substantial material, either solid, or with openings not over one inch in greatest dimension. (ii) Where material is piled to such height that a standard toe board does not provide protection, paneling, or screening from floor to intermediate rail or to top rail must be provided.	Toe boards are not applicable as railing to be mounted on top of walls, which will be constructed with top of wall 12" above grade. Additionally chain link mesh will be provided on full face.

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Develop loading on panels above grade - assume to behave similar to noise wall.

WSDOT BDM, Chapter 8.2 Noise Barrier Walls

8.2.2 Loads

Noise barrier walls and their components shall be designed for all applicable loads defined in the current AASHTO LRFD Chapter 3.

Wind loads and on noise barriers shall be as specified in Chapter 3.

Seismic load shall be as follows:

The effect of earthquake loading on noise barrier walls shall be investigated using the Extreme Event I limit states of AASHTO LRFD Table 3.4.1-1 with the load factor $\gamma_p = 1.0$.

Seismic loads shall be taken to be horizontal design force effects determined in accordance with the AASHTO LRFD provisions of Article 4.7.4.3.3 on the basis of the elastic response coefficient, C_{sm}, specified in Article 3.10.4 and BDM Section 4, and the dead load of sound barrier. The seismic design force effects for connections shall be determined by dividing the force effects resulting from elastic analysis by the response modification factor, R, specified in Table 8.2-1.

Table 8.2-1 Response Modification Factors, R

Connection	R
Monolithic connection	1.0
Connection of precast wall to bridge barrier	0.3
Connection of precast wall to retaining wall or moment slab barrier	0.5
Connection of precast wall to shaft	0.8

WSDOT BDM, Appendix 8.1-A1

Non-Standard Noise Barrier Walls	General	Design shall be based on current editions, including current interims, of the following documents; AASHTO LRFD Bridge Design Specifications, WSDOT GDM and WSDOT BDM.
	Seismic	AASHTO Guide Specifications for LRFD Seismic Bridge Design using the USGS 2014 Seismic Hazard Maps with Seven Percent Probability of Exceedance in 75 yrs (1000 yr Return Period) and the site coefficients or site-specific procedure provided in the WSDOT Design Memorandum dated January 8, 2017 and WSDOT BDM Chapter 4.
	Traffic Barrier	WSDOT BDM and the AASHTO *LRFD Bridge Design Specifications* Section A13.3 for Concrete Railings considering a minimum TL-4 impact load.

WSDOT BDM, 3.11 Wind Loads

3.11.3 Wind on Noise Walls

Wind on Noise Walls shall be as specified in LRFD 3.8.1, 3.8.1.2.4, and 15.8.2.

AASHTO Section 15 Design of Sound Barriers:

15.8.2—Wind Load

The provisions of Article 3.8.1 shall apply.

C15.8.2

The wind load provisions included in Article 3.8.1 are applicable to all structures, including sound barriers. This deviates from earlier specifications where special wind provisions for sound barriers were included.

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3.8.1.2.4—Wind Loads on Sound Barriers

The wind pressure on ground-mounted or structure-mounted sound barriers shall be determined using Eq. 3.8.1.2.1-1 and assuming the wind direction perpendicular to the plane of the sound barrier.

The sound barrier panels shall be designed assuming the wind pressure is applied as a uniform load to the entire area of the panels.

The vertical support elements (if used), the foundations, and the connection of the panel or the vertical support elements to the foundations or the supporting structure shall be designed for a line load equal in value to the wind pressure multiplied by the sound barrier height. The line load shall be applied at a distance equal to 0.55 times the sound barrier height measured from the bottom of the sound barrier. For determining the location of the line load, the height of the sound barrier shall be taken as the distance from the top of the sound barrier to:

- The ground surface immediately adjacent to the sound barrier for ground-mounted sound barriers.
- The elevation of the sound barrier connection to the supporting structure for structure-mounted sound barriers.

Where the sound barrier is mounted on top of a traffic railing or a retaining wall extending above ground, the magnitude and location of the wind loads transmitted to the base of the supporting traffic railing or retaining wall shall be determined as specified above, assuming that the height of the exposed area is the sum of the height of the sound barrier plus the height of the supporting railing or retaining wall.

The height of the supporting railing or retaining wall to be considered in determining the magnitude and location of the wind load shall be that measured from the top surface of the ground, bridge deck, or roadway pavement to the top of the supporting railing or retaining wall.

C3.8.1.2.4

The wind pressure is applied as a constant pressure over the entire area of the sound barrier. In reality, the wind speed and, consequently, the wind pressure, increase with the increase in height above the surrounding ground surface. Applying the wind load as a line load at a location above mid-height of the sound barrier better reflects the effect of the uneven pressure distribution.

Where the ground surface elevation is not the same in the front and in the back of a ground-mounted sound barrier, the wind forces will need to be determined for each direction as a separate case of loading. The design of all components must satisfy the demand from both cases.

3.8.1.2—Wind Load on Structures: *WS*

3.8.1.2.1—General

The wind pressure shall be determined as:

$$P_Z = 2.56 \times 10^{-6} V^2 K_z G C_D \qquad (3.8.1.2.1\text{-}1)$$

where:

P_Z = design wind pressure (ksf)

V = design 3-second gust wind speed specified in Table 3.8.1.1.2-1 (mph)

K_Z = pressure exposure and elevation coefficient to be taken equal to K_Z (B), K_Z (C), or K_Z (D) determined using Eqs. 3.8.1.2.1-2, 3.8.1.2.1-3, or 3.8.1.2.1-4, respectively, for Strength III and Service IV load combinations and to be taken as 1.0 for other load combinations

G = gust effect factor determined using a structure-specific study or as specified in Table 3.8.1.2.1-1 for Strength III and Service IV load combinations and 1.0 for other load combinations

C_D = drag coefficient determined using a structure-specific study or as specified in Table 3.8.1.2.1-2

C3.8.1.2.1

The basis for the development of wind load provisions exists in Wassef and Raggett (2014).

For structure heights less than 33.0 ft, the proximity to the ground surface causes turbulence for which the effect on wind pressure cannot be accurately determined. Therefore, no reduction in the value of K_Z is shown in Table C3.8.1.2.1-1 for structure heights less than 33.0 ft.

Strength V and Service I load combinations are based on constant wind speeds that are not functions of the bridge type, bridge height, or the wind exposure category at the location of the bridge. Therefore, the pressure exposure and elevation coefficient, K_Z, is taken as 1.0 for these load combinations.

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When the wind speed, K_Z, and G specified for Strength V and Service I load combinations are substituted in Eq. 3.8.1.2.1-1, the resulting wind pressure on bridge structures, P_Z, becomes a multiple of the drag coefficient, C_D, for the structure being considered. The wind pressure in these cases may be calculated using Table C3.8.1.2.1-2.

Table C3.8.1.2.1-2—Wind Pressure on the Bridge Structures for Strength V and Service I Load Combinations

Load Combination	Wind Pressure on the Structure, P_Z, for the Specified Wind Speed (ksf)
Strength V	0.0163 C_D
Service I	0.0125 C_D

Table 3.8.1.2.1-1—Gust Effect Factor, *G*

Structure Type	Gust Effect Factor, *G*
Sound Barriers	0.85
All other structures	1.00

Table C3.8.1.2.1-1—Pressure Exposure and Elevation Coefficients, K_Z

Structure Height, *Z* (ft)	Wind Exposure Category B	Wind Exposure Category C	Wind Exposure Category D
≤ 33	0.71	1.00	1.15
40	0.75	1.05	1.20
50	0.81	1.10	1.25
60	0.85	1.14	1.29
70	0.89	1.18	1.32
80	0.92	1.21	1.35
90	0.95	1.24	1.38
100	0.98	1.27	1.41
120	1.03	1.32	1.45
140	1.07	1.36	1.49
160	1.11	1.40	1.52
180	1.15	1.43	1.55
200	1.18	1.46	1.58
250	1.24	1.52	1.63
300	1.30	1.57	1.68

Table 3.8.1.2.1-2—Drag Coefficient, C_D

Component		Drag Coefficient, C_D Windward	Leeward
I-Girder and Box-Girder Bridge Superstructures		1.3	N/A
Trusses, Columns, and Arches	Sharp-Edged Member	2.0	1.0
	Round Member	1.0	0.5
Bridge Substructure		1.6	N/A
Sound Barriers		1.2	N/A

Table 3.8.1.1.2-1—Design 3-Second Gust Wind Speed for Different Load Combinations, *V*

Load Combination	3-Second Gust Wind Speed (mph), *V*
Strength III	Wind speed taken from Figure 3.8.1.1.2-1
Strength V	80
Service I	70
Service IV	0.75 of the speed used for the Strength III limit state

Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).

Figure 3.8.1.1.2-1—Design Wind Speed, *V*, in mph (m/s)

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3.8.1.1.4—Ground Surface Roughness Categories

A ground surface roughness within each of the 45 degree sectors defined in Article 3.8.1.1.3 shall be determined as follows:

- Ground Surface Roughness B: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger;
- Ground Surface Roughness C: Open terrain with scattered obstructions having heights generally less than 33.0 ft, including flat open country and grasslands; and
- Ground Surface Roughness D: Flat, unobstructed areas and water surfaces; this category includes smooth mud flats, salt flats, and unbroken ice.

3.8.1.1.5—Wind Exposure Categories

The exposure category of the structure shall be determined as follows:

- Wind Exposure Category B: Wind Exposure Category B shall apply where the Ground Surface Roughness Category B, as defined in Article 3.8.1.1.4, prevails in the upwind direction for a distance greater than 1,500 ft for structures with a mean height of less than or equal to 33 ft, and for a distance greater than 2,600 ft or 20 times the height of the structure, whichever is greater, for structures with a mean height greater than 33 ft.
- Wind Exposure Category C: Wind Exposure Category C shall apply for all cases where Wind Exposure Categories B or D do not apply.
- Wind Exposure Category D: Wind Exposure Category D shall apply where the Ground Surface Roughness Category D, as defined in Article 3.8.1.1.4, prevails in the upwind direction for a distance greater than 5,000 ft or 20 times the height of the structure, whichever is greater. Wind Exposure Category D shall also apply where the structure is within a distance of 600 ft or 20 times the height of the structure, whichever is greater, from a Ground Surface Roughness Category D condition, even if Ground Surface Roughness Category B or C exist immediately upwind of the structure.

For this wall location, by highway on hill - assume:

Ground Surface Roughness C

Wind Exposed Category C

<u>Develop Strength III Wind Load - per AASHTO LRFD section above (use alone as loading):</u>

$P_Z = 2.56x10^{-6} * V^2 * Kz * G * C_D$

V =	110	mph	(figure 3.8.1.1.2-1)
Kz =	1		(table C3.8.1.2.1-1, Z </= 33 ft - design for top of panel backfill side to first nail)
G =	0.85		(table 3.8.1.2.1-1)
C_D =	1.2		(table 3.8.1.2.1-2)
P_Z =	**0.0316**	**ksf**	

<u>Develop Strength V Wind Load - per AASHTO LRFD section above (use in combination with railing loading):</u>

$P_Z = 2.56x10^{-6} * V^2 * Kz * G * C_D$

V =	80	mph	(table 3.8.1.1.2-1)
Kz =	1		(for Strength V)
G =	1		(for Strength V)
C_D =	1.2		(table 3.8.1.2.1-2)
P_Z =	**0.0197**	**ksf**	

<u>Develop Service I Wind Load - per AASHTO LRFD section above (use alone as loading):</u>

$P_Z = 0.0125 * C_D$			(table C3.8.1.2.1-2)
C_D =	1.2		(table 3.8.1.2.1-2)
P_Z =	**0.015**	**ksf**	

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AASHTO Section 4 Structural Analysis and Evaluation (see separate calculations for development of seismic forces)

4.7.4.3.3—Multimode Spectral Method

The multimode spectral analysis method shall be used for bridges in which coupling occurs in more than one of the three coordinate directions within each mode of vibration. As a minimum, linear dynamic analysis using a three-dimensional model shall be used to represent the structure.

The number of modes included in the analysis should be at least three times the number of spans in the model. The design seismic response spectrum as specified in Article 3.10.4 shall be used for each mode.

The member forces and displacements may be estimated by combining the respective response quantities (moment, force, displacement, or relative displacement) from the individual modes by the Complete Quadratic Combination (CQC) method.

C4.7.4.3.3

Member forces and displacements obtained using the CQC combination method are generally adequate for most bridge systems (Wilson et al., 1981).

If the CQC method is not readily available, alternative methods include the square root of the sum of the squares method (SRSS), but this method is best suited for combining responses from well-separated modes. For closely spaced modes the absolute sum of the modal responses should be used.

AASHTO Section 3 Loads and Load Factors

3.10.4.2—Elastic Seismic Response Coefficient

For periods less than or equal to T_0, the elastic seismic coefficient for the mth move of vibration, C_{sm}, shall be taken as:

$$C_{sm} = A_S + (S_{DS} - A_S)\,(T_m / T_0) \qquad (3.10.4.2\text{-}1)$$

in which:

$$A_S = F_{pga}\,PGA \qquad (3.10.4.2\text{-}2)$$

$$S_{DS} = F_a\,S_S \qquad (3.10.4.2\text{-}3)$$

where:

- PGA = peak ground acceleration coefficient on rock (Site Class B)
- S_S = horizontal response spectral acceleration coefficient at 0.2-sec period on rock (Site Class B)
- T_m = period of vibration of mth mode (s)
- T_0 = reference period used to define spectral shape = $0.2\ T_S$ (s)
- T_S = corner period at which spectrum changes from being independent of period to being inversely proportional to period = S_{D1}/S_{DS} (s)

For periods greater than or equal to T_0 and less than or equal to T_S, the elastic seismic response coefficient shall be taken as:

$$C_{sm} = S_{DS} \qquad (3.10.4.2\text{-}4)$$

For periods greater than T_S, the elastic seismic response coefficient shall be taken as:

$$C_{sm} = S_{D1} / T_m \qquad (3.10.4.2\text{-}5)$$

in which:

$$S_{D1} = F_v\,S_1 \qquad (3.10.4.2\text{-}6)$$

where:

- S_1 = horizontal response spectral acceleration coefficient at 1.0 sec period on rock (Site Class B)

C3.10.4.2

An earthquake may excite several modes of vibration in a bridge and, therefore, the elastic response coefficient should be found for each relevant mode.

The discussion of the single-mode method in the commentary to Article 4.7.4.3.2 illustrates the relationship between period, C_{sm}, and quasi-static seismic forces, $p_e(x)$. The structure is analyzed for these seismic forces in the single-mode method. In the multimode method, the structure is analyzed for several sets of seismic forces, each corresponding to the period and mode shape of one of the modes of vibration, and the results are combined using acceptable methods, such as the Complete Quadratic Combination method as required in Article 4.7.4.3.3. C_{sm} applies to weight, not mass.

Table 4: Segment 2A Seismic Design Parameters

Parameter	Return Period
	1,000-year (SEE)
Site class	C
Peak ground acceleration (PGA)	0.431g
F_{PGA}	1.200
Site-adjusted peak ground acceleration (A_S)	0.517g
Short-period (0.2-second) spectral acceleration (S_S)	0.98g
Site coefficient (Fa)	1.200
Short-period design response acceleration (S_{DS}) = S_S x Fa	1.176g
1-second period spectral acceleration (S_1)	0.283g
Site coefficient (Fv)	1.500
1-second design response acceleration S_{D1} = S_1 x Fv	0.425g
Mean earthquake magnitude (Mw)	7.0

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Facing Design - Section A-A

Beginning of Wall (South End)

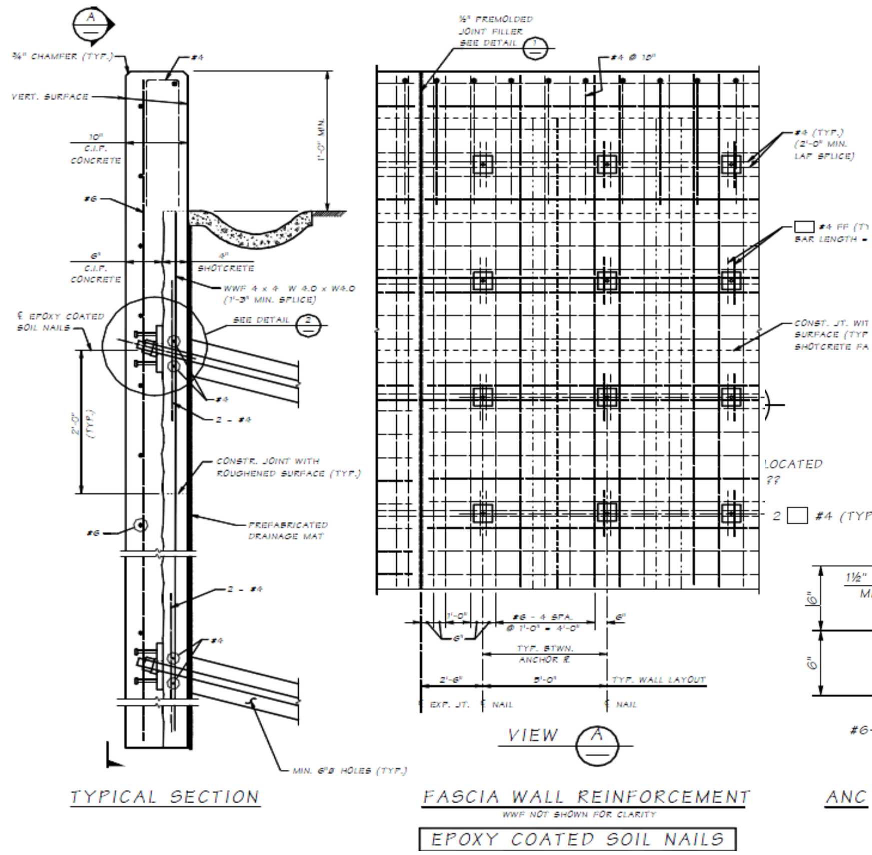
Station 1+00 (start) to 2+20

L = 15'

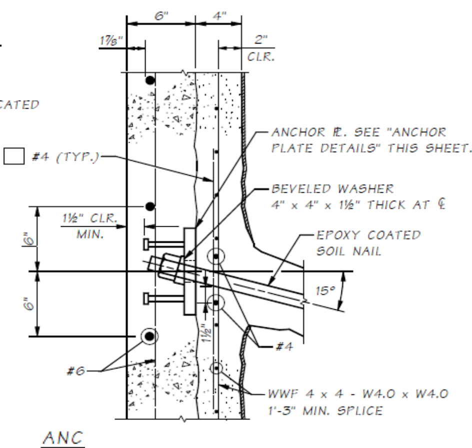
DTL = 1,700 plf for Top Row; 2,500 plf for Remaining Rows

Nail Head Force: 18 kips static; 36 kips seismic

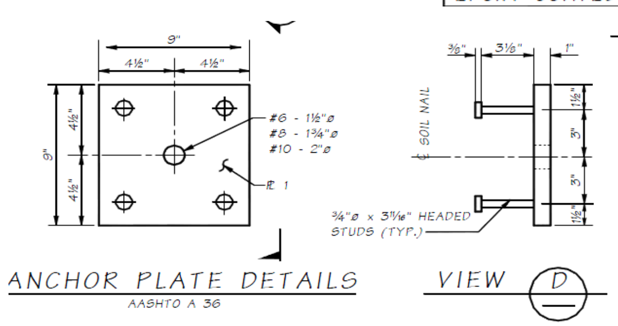
Will use BDM Standard Sheet for dimensions and confirm with analysis along with spacing and nail information provided by Geotech:

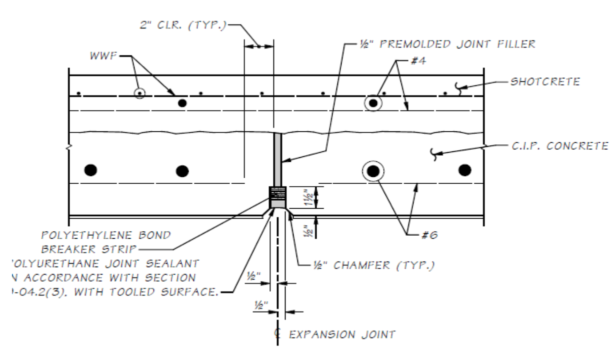


Per FHWA - need min 2" CLR for studs for corrosion protection - all walls will have additional ~1-1.5" of formliner - therefore will keep 1.5" clr for calculations.



Note that L_S and h_H must be selected such that the cover of final shotcrete or concrete over the stud is at least 2 in. according to Section 7.6.7 "Nail-Head Corrosion Protection." Available sizes of headed studs can be found in Appendix A and in references provided by manufacturers.





From FHWA NHI 14-007/GEC No. 7:

D. Facing Design
1) Nail head load
2) Wall facing type and thickness
3) Facing materials
4) Flexural resistance
5) Facing punching shear resistance
6) Facing head stud resistance

The safety factors correspond to the potential failure modes of the nail-facing connection including the flexural and punching shear failures. Because a two-phase facing construction is used in this project, flexural and shear-punching failure modes must be evaluated separately for the temporary and the permanent facing. Additionally, for the final facing, a tensile failure of the headed studs is considered.

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Facing Design - Section A-A

Station 1+00 (start) to 2+20

Provided:

Nail Size = #8
Grade = 75.00 ksi
S_H = 6.00 ft
S_V = 5.00 ft

Nail Head Force: 18 kips static; 36 kips seismic

Loads provided are from Geotech and have been converted to To (design nail tensile force).

Step 1 - Calculate the Design Nail Head Tensile Force - Provided by Geotechs (did not need to compute)

Permanent Static:

To = 18.00 kips
γ_{EV} = 1.35 (AASHTO LRFD, Table 3.4.1-2, EV for retaining walls, per FHWA)

Seismic:

To = 36.00 kips
Y = 1.00 (AASHTO LRFD, Table 3.4.1-1, Extreme Event I)

Step 2 - Select Wall Facing Thickness:

shotcrete thickness (h) = 6.00 in (initial)
CIP Fascia thickness (h) = 9.00 in (final)

Step 3 - Select Soil Wall Materials:

Bar Nail fy = 75.00 ksi (provided by Geotech)

Initial Facing - Shotcrete

WWF fy = 60.0 ksi
f'c = 4.0 ksi Specifications 6-18.3(2), shotcrete f'c=4000 psi
WWF = 4x4 - W4.0 x W4.0
a_{hm}, a_{vm} = 0.12 in^2 /ft
Bars at Nails = 4 (2 vert, 2 horz)
Bar As = 0.20 in^2
No bars vert = 2
A_{VN} = 0.40 in^2
No bars horz = 2
A_{HN} = 0.40 in^2

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Facing Design - Section A-A

Station 1+00 (start) to 2+20

Perm Facing - CIP Fascia

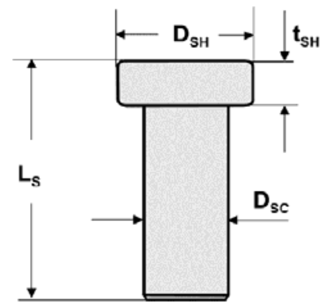
Rebar fy =	60.0	ksi
CIP bars =	5	
Bar As =	0.31	in^2 / bar
spacing =	12.0	in
f'c =	4.0	ksi

TABLE A.6
HEADED-STUD DIMENSIONS

Headed-Stud Size	Nominal Length L_s		Head Diameter D_H		Shaft Diameter D_S		Head Thickness t_H		Head Area/ Shaft Area	Head Thickness/ (Head Diameter-Shaft Diameter)
	mm	in.	mm	in.	mm	in.	mm	in.	in.	
$^3/_4$ x $3^{11}/_{16}$	89	15.5	31.8	1.3	19.1	0.750	9.5	0.38	2.8	0.75

Bearing Plate/Anchors

L_{BP} =	9.0	in	
W_{BP} =	9.0	in	
tp =	1.0	in	
fy =	36.00	ksi	Grade ASTM A36
no. of studs N_H =	4		(per BDM example)
headed stud dia D_{SC} =	0.75	in	
stud shaft area As =	0.44	in^2	
head size D_{SH} =	1.30	in	
clr =	1.50	in	
shaft L =	6.13	in	
head t_{SH} =	0.375	in	(per BDM example)
Ls = shaft L + t_{SH} =	6.50	in	(max for CIP thickness)
S_{SH}, spa =	6.00	in	
fy =	60.00	ksi	A307, Grade A



Step 4 - Flexural Resistance:

The facing resistance is verified in LRFD for each of the facings as follows:

$$CDR = \frac{\phi_{FF}\ R_{FF}}{\gamma\ T_o} \geq 1.0$$

Equation 6.18: Capacity-to-demand ratio for bending in facing.

Where:

ϕ_{FF} = resistance factor for bending/flexure in the facing

R_{FF} = nominal resistance for bending/flexure of facing

γ = load factor selected for verifications

T_o = maximum tensile force at soil nail head, as estimated with Eq. 5.1 (Section 5.2.1)

If the resistance is insufficient, increase the thickness of facing, amount of steel, and/or strength of steel and/or of concrete.

5.8.3c Resistance Factors for Flexure at Facing

Resistance factors for flexure resistance at the facing are back-calibrated for a load factor of γ_{EV} = 1.35 for static conditions and γ_{EV} = 1.00 for seismic loading to match ASD-based designs. These values are presented in Table 5.9.

Table 5.9: Resistance Factors for Flexure Resistance at Facing

Condition	Case	Symbol	Resistance Factor
Static	Initial and final facing	ϕ_{FF}	0.90
Seismic loading	Initial and final facing	ϕ_{FF}	0.90

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Facing Design - Section A-A

Station 1+00 (start) to 2+20

$$R_{FF}\,[\text{kip}] = 3.8 \times C_F \times f_y\,[\text{ksi}] \times F = 228 \times C_F \times F$$

Equation 6.15: Nominal bending resistance for Grade 60 steel WWM/rebar and 4,000 psi shotcrete.

Where:

$$F = \text{smaller of} \begin{cases} (a_{vn} + a_{vm})[\text{in}^2/\text{ft}] \times \left(\dfrac{S_H\ h[\text{ft}]}{S_V}\right) \\ (a_{hn} + a_{hm})[\text{in}^2/\text{ft}] \times \left(\dfrac{S_V\ h[\text{ft}]}{S_H}\right) \end{cases}$$

Equation 6.16: Definition of F.

Table 6.4: Nomenclature for Cross-Sectional Area per Unit Width of Facing Reinforcement

Direction	Location	Cross-Sectional Area of Reinforcement per Unit Width
Vertical	Nail Head (1)	a_{vn}
Vertical	Mid-Span (2)	a_{vm}
Horizontal	Nail Head (1)	a_{hn}
Horizontal	Mid-Span (2)	a_{hm}

Notes: (1) Both WWM and the rebar contribute.
(2) Only WWM contributes.

Table 6.5: Factor C_F

Facing	Facing Thickness, h_i or h_f (in.)	C_F
Initial	4	2.0
Initial	6	1.5
Initial	8	1.0
Final	All	1.0

Where:

a_{vn} = reinforcement cross-sectional area per unit width in the vertical direction at the nail head. If the yield strengths of the WWM and the rebar are different in the initial facing, this is an equivalent value.

a_{vm} = reinforcement cross-sectional area per unit width in the vertical direction at midspan

a_{hn} = equivalent reinforcement cross-sectional area per unit width in the horizontal direction at the nail head

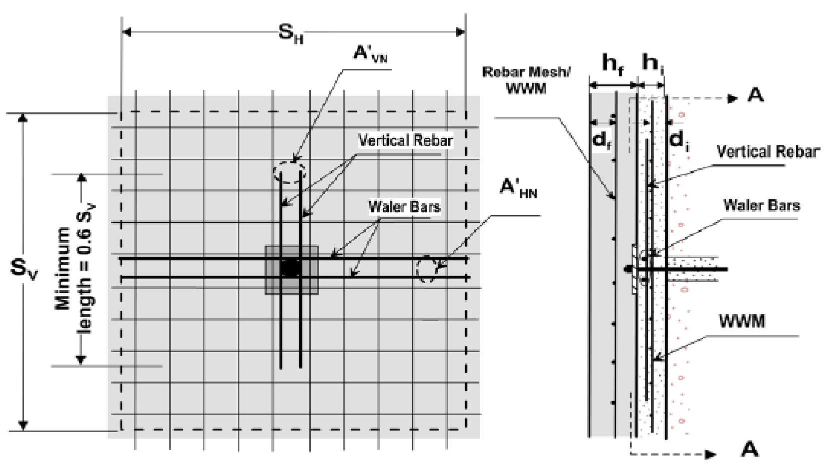
a_{hm} = reinforcement cross-sectional area per unit width in the horizontal direction at midspan

A'_{VN} = equivalent cross-sectional area at the head in the vertical direction to consider different yield strengths for the WWM and rebar. $A'_{VN} = (f_{y\_R}/f_{y\_W}) \times A_{VN}$, where f_{y_R} = rebar yield strengths and f_{y_W} = WWM yield strength

A'_{HN} = equivalent cross-sectional area at the head in the horizontal direction. $A'_{HN} = (f_{y\_R}/f_{y\_W}) \times A_{HN}$

S_H = nail horizontal spacing

S_V = nail vertical spacing



Section A-A

Vertical Cross Sectional Area (per unit length)

Horizontal Cross Sectional Area (per unit length)

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Facing Design - Section A-A

Station 1+00 (start) to 2+20

Initial Facing (Shotcrete)

	a_{vm} = 0.12 in^2	a_{hm} = 0.12 in^2
	$a_{vn} = a_{vm} + A_{VN}/S_H$	$a_{hn} = a_{hm} + A_{HN}/S_V$
	A_{VN} = 0.40 in^2	A_{HN} = 0.40 in^2
	S_H = 6.0 ft	S_V = 5.0 ft
	a_{vn} = 0.19 in^2 /ft	a_{hn} = 0.20 in^2 /ft
	$F = (a_{vn} + a_{vm}) * S_H/S_V*h$	$F = (a_{hn} + a_{hm}) * S_V/S_H*h$
	h = 0.50 ft	h = 0.50 ft
F = smaller of:	F = 0.18	**F = 0.13**

$R_{FF} = 3.8*C_F*fy*F$

C_F = 1.50 (Table 6.5, for initial, thickness = 6.00 inches)

R_{FF} = 45.60 kips

$CDR = \phi_{FF}*R_{FF}/\gamma*To$

ϕ_{FF} = 0.90 (Table 5.9)

Static: γ_{EV} = 1.35

To = 18.00 kips

Flexural Resistance **CDR = 1.69** >= 1.00 OK

Seismic γ = 1.00

To = 36.00 kips

Flexural Resistance **CDR = 1.14** >= 1.00 OK

Check reinforcement ratios:

$$\rho_{TOT}(\%) = \left[\frac{a_{vn} + a_{vm}}{12\left(\frac{h_i}{2}\right)}\right] \times 100$$

(horizontal) ρ_{TOT} = 0.89 %

(vertical) ρ_{TOT} = 0.85 %

Calculate the reinforcement ratio, ρ_{ij}, as:

$$\rho_{ij} = \frac{a_{ij}}{0.5\,h}\ 100$$

Equation 6.6: Reinforcement ratio.

Where, a_{ij} = ratio of cross-sectional area of reinforcement per unit width (in "i" direction and "j" location) and h = thickness of the facing being designed, whether initial or final. The direction "i" can be "v" (for vertical) or "h" (for horizontal); the location "j" can be "n" (nail head) or "m" (mid-span between nails).

$$\rho_{min}[\%] = 0.24\frac{\sqrt{f'_c[psi]}}{f_y[ksi]}$$

$$\rho_{max}[\%] = 0.05\frac{f'_c[psi]}{f_y[ksi]}\left(\frac{90}{90 + f_y[ksi]}\right)$$

	ρ_{min} = 0.25 %	ρ_{max} = 2.00 %
(horizontal)	< 0.89 OK	> 0.89 OK
(vertical)	< 0.85 OK	> 0.85 OK

The additional horizontal and vertical rebar in the initial facing must be placed near the nail heads, usually within a distance h_i from the walls of the drill hole. Horizontal bars should have a length of at least $0.60S_H$; however, it is not uncommon to place waler bars with a length equal to S_H (Figure 6.5). Vertical rebar should be at least $0.60S_V$ long. Laps between adjacent bars must be in the mid-span between nails. The wire spacing in WWM typically used in soil nail walls must meet the requirements for maximum spacing of reinforcement per Section 10.6 "Distribution of Flexural Reinforcement" of ACI (2011).

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Facing Design - Section A-A

Station 1+00 (start) to 2+20

Final Facing (CIP Fascia)

	$a_{vm} = a_{vn} =$	0.31	in^2	$a_{hm} = a_{hn} =$	0.31 in^2
	$S_H =$	6.0	ft	$S_V =$	5.0 ft
	$F = (a_{vn} + a_{vm}) * S_H/S_V*h$			$F = (a_{hn} + a_{hm}) * S_V/S_H*h$	
	h =	0.75	ft	h =	0.75 ft
F = smaller of:	F =	0.56		**F =**	**0.39**

$R_{FF} = 3.8*C_F*fy*F$

$C_F =$ 1 (Table 6.5, for final)

$R_{FF} =$ 88.35 kips

$CDR = \phi_{FF}*R_{FF}/\gamma*To$

$\phi_{FF} =$ 0.90 (Table 5.9)

Static: $\gamma_{EV} =$ 1.35

To = 18.00 kips

Flexural Resistance **CDR = 3.27** **>= 1.00 OK**

Seismic: $\gamma =$ 1.00

To = 36.00 kips

Flexural Resistance **CDR = 1.64** **>= 1.00 OK**

Check reinforcement ratios:

$$\rho_{TOT}\ (\%) = \left[\frac{a_{vn} + a_{vm}}{12\left(\frac{h_i}{2}\right)}\right] \times 100$$

(horizontal) $\rho_{TOT} =$ 1.15 %

(vertical) $\rho_{TOT} =$ 1.15 %

Calculate the reinforcement ratio, ρ_{ij}, as:

$$\rho_{ij} = \frac{a_{ij}}{0.5\,h}\ 100$$

Equation 6.6: Reinforcement ratio.

Where, a_{ij} = ratio of cross-sectional area of reinforcement per unit width (in "i" direction and "j" location) and h = thickness of the facing being designed, whether initial or final. The direction "i" can be "v" (for vertical) or "h" (for horizontal); the location "i" can be "n" (nail head) or "m" (mid-span between nails).

$$\rho_{min}[\%] = 0.24\frac{\sqrt{f'_c[psi]}}{f_y[ksi]}$$

$$\rho_{max}[\%] = 0.05\frac{f'_c[psi]}{f_y[ksi]}\left(\frac{90}{90 + f_y[ksi]}\right)$$

	$\rho_{min} =$	0.25 %		$\rho_{max} =$	2.00 %
(horizontal)	**<**	**1.15**	**OK**	**>**	**1.15 OK**
(vertical)	**<**	**1.15**	**OK**	**>**	**1.15 OK**

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Facing Design - Section A-A

Station 1+00 (start) to 2+20

Step 5 - Facing Punching Shear Resistance:

The nominal facing punching shear resistance, R_{FP}, for either situation must meet the following condition:

$$CDR = \frac{\phi_{FP} \ R_{FP}}{\gamma \ T_o} \geq 1.0$$

Equation 6.19: Capacity-to-demand ratio for punching shear resistance.

Where:

ϕ_{FP} = resistance factor for punching shear in the facing

γ = load factor selected for verification

T_o = maximum tensile force at soil nail head, as defined previously

R_{FP} is estimated as:

$$R_{FP} = C_P \ V_F$$

Equation 6.20: Nominal punching shear resistance at facing.

Where:

C_P = dimensionless factor that accounts for the contribution of the soil support under the nail head to the shear resistance

V_F = concrete punching shear basic resistance acting through the facing section

5.8.4 Punching Shear Strength Limit State

Resistance factors for punching at the facing are back-calibrated for a load factor of $\gamma_{EV} = 1.35$ for static conditions and $\gamma_{EV} = 1.00$ for seismic loading to match ASD-based designs. These values are presented in Table 5.10.

Table 5.10: Resistance Factors for Punching Shear at Facing

Condition	Case	Symbol	Resistance Factor
Static	Initial and final facing	ϕ_{FP}	0.90
Seismic loading	Initial and final facing	ϕ_{FP}	0.90

C_P can be as high as 1.15 if the soil reaction is considered. The contribution from the soil support behind the wall is conservatively assumed to be negligible; therefore, $C_P = 1.0$. Tl

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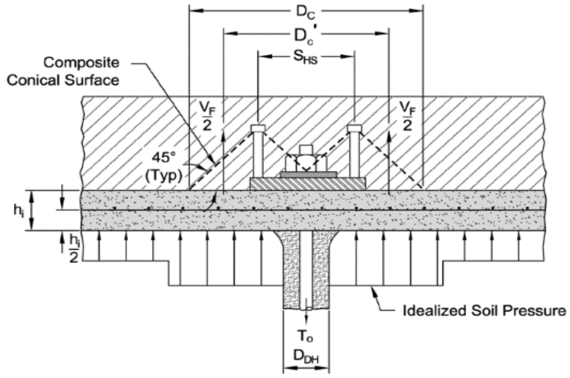
Facing Design - Section A-A

Station 1+00 (start) to 2+20

$$V_F\,[\text{kip}] = 0.58\sqrt{f'_c\ [\text{psi}]}\ \pi\ D'_c\,[\text{ft}]\,h_c\,[\text{ft}]$$

Equation 6.21: Nominal punching shear resistance through facing.



(b) Headed Stud Connection

Initial Facing (Shotcrete)

$D'c = L_{BP} + h_i$
$L_{BP} =$ 9.00 in
$hc = h_i =$ 6.00 in
$D'c =$ 15.00 in
$V_F =$ 71.99 kips

$C_P =$ 1.00 (assume negligible)
$R_{FP} = C_P*V_F =$ 71.99 kips

$CDR = \phi_{FP}*R_{FP}/\gamma*To$
$\phi_{FP} =$ 0.90 (Table 5.10)

Static: $\gamma_{EV} =$ 1.35
To = 18.00 kips
Punching Shear **CDR = 2.67** >= 1.00 OK

Seismic: $\gamma =$ 1.00
To = 36.00 kips
Punching Shear **CDR = 1.33** >= 1.00 OK

Final Facing (CIP Fascia)

$D'c =$ smaller $S_{SH}+h_c$ or $2*h_c$
$hc = L_S + tp - t_{SH}$
$L_S =$ 6.50 in
tp = 1.0 in
$t_{SH} =$ 0.375 in
hc = 7.125 in
$S_{SH} =$ 6.00 in
$S_{SH}+h_c =$ 13.125 in
$2*h_c =$ 14.25 in
D'c (min) = 13.125 in
$V_F =$ 74.80 kips

$C_P =$ 1.00 (assume negligible)
$R_{FP} = C_P*V_F =$ 74.80 kips

$CDR = \phi_{FP}*R_{FP}/\gamma*To$
$\phi_{FP} =$ 0.90 (Table 5.9)

Static: $\gamma_{EV} =$ 1.35
To = 18.00 kips
Punching Shear **CDR = 2.77** >= 1.00 OK

Seismic: $\gamma =$ 1.00
To = 36.00 kips
Punching Shear **CDR = 1.39** >= 1.00 OK

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Facing Design - Section A-A

Station 1+00 (start) to 2+20

Step 6 - Facing Head Stud Resistance:

$$CDR = \frac{\phi_{FH}\ R_{FH}}{\gamma\ T_o} \geq 1.0$$

Equation 6.27: Capacity-to-demand ratio for a headed stud.

Where:

ϕ_{FH} = resistance factor for headed stud tensile resistance
R_{FH} = nominal tensile resistance of headed studs in final facings
γ = load factor selected for verification
T_o = as defined earlier

$$R_{FH} = N_H\ A_S\ f_{y\text{-}hs}$$

Equation 6.26: Headed stud resistance at facing.

Where:

N_H = number of headed studs in the connection (usually 4)
A_S = cross-sectional area of the headed stud shaft
$f_{y\text{-}hs}$ = tensile yield strength of headed stud

5.8.5 Headed Stud in Tension in Final Facing

Resistance factors for headed stud in tension punching in the final facing are back-calibrated for a load factor of γ_{EV} = 1.35 for static conditions and γ_{EV} = 1.00 for seismic loading to match ASD-based designs. These values are presented in Table 5.11.

Table 5.11: Resistance Factors for Headed Stud in Tension in Final Facings

Condition	Case	Symbol	Resistance Factor
Static	A307 Steel Bolt (1)	ϕ_{FH}	0.70
Static	A325 Steel Bolt	ϕ_{FH}	0.80
Seismic loading	A307 Steel Bolt (1)	ϕ_{FH}	0.65
Seismic loading	A325 Steel Bolt	ϕ_{FH}	0.75

Note: (1) This is equivalent to AWS D1.1 Type B studs, with f_y = 60 ksi.

N_H = 4
As = 0.44 in^2
fy = 60.00 ksi
R_{FH} = 105.98 kips

CDR = ϕ_{FP}*R_{FP}/γ*To

ϕ_{FP} = 0.70 (Table 5.11)

Static:
γ_{EV} = 1.35
To = 18.00 kips
Stud Resistance **CDR = 3.05** **>= 1.00 OK**

Seismic:
γ = 1.00
To = 36.00 kips
Stud Resistance **CDR = 1.53** **>= 1.00 OK**

Summary

Shotcrete Layer:
thickness = 6.00 in
main reinf = 4x4 - W4.0 x W4.0 — Horz bar L = 0.6Sh = 3.60 ft
bar size at nails = 4 (2 vert and 2 horz) — Vert bar L = 0.6Sv = 3.00 ft

CIP Fascia
thickness = 9.00 in
bar size 5 spaced at 12.0 in

Bearing Plate
square - size = 9.0 x 9.0 x 1.0 in thick
4 anchors spaced at = 6.00 in
anchor L = 6.50 in
anchor D = 0.75 in

DOES NOT GOVERN

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BY	JMB	DATE	11/23/2021		
CHECKED BY	DMP	DATE	11/23/2021		

wood.
Colorado Center Tower II
2000 S. Colorado Blvd., Ste 2-1000
Denver, CO 80222
+1 (303) 935-6505 Fax '+1 (303) 935-6575

Facing Design - Section B-B

Middle Portion of Wall

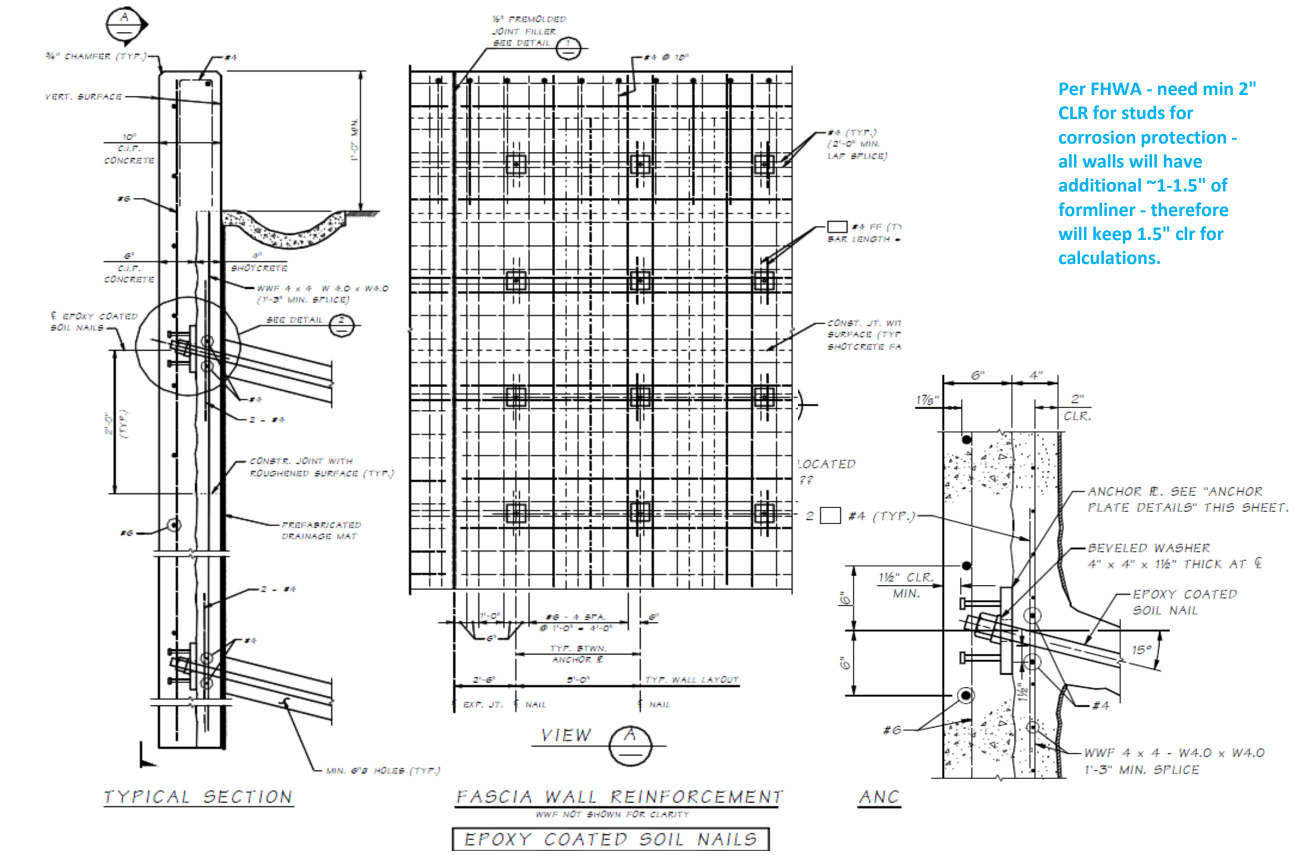
Station 2+20 to 5+20

L = 25'

DTL = 1,700 plf for Top 3 Rows; 2,500 plf for Remaining Rows

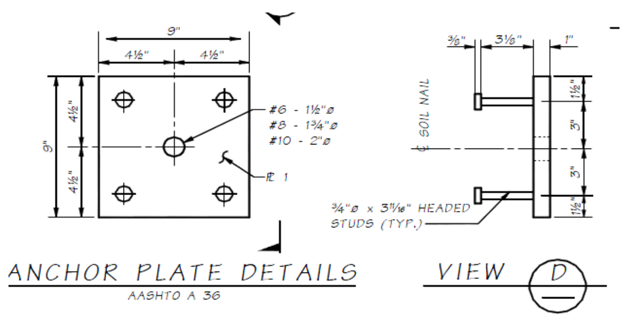
Nail Head Force: 33 kips static; 32 kips seismic

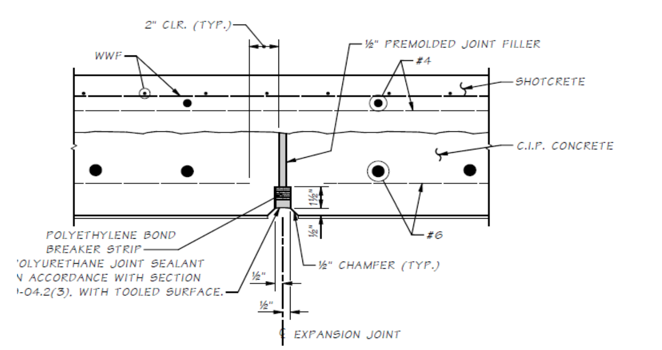
Will use BDM Standard Sheet for dimensions and confirm with analysis along with spacing and nail information provided by Geotech:



Per FHWA - need min 2" CLR for studs for corrosion protection - all walls will have additional ~1-1.5" of formliner - therefore will keep 1.5" clr for calculations.

Note that L_S and h_t must be selected such that the cover of final shotcrete or concrete over the stud is at least 2 in. according to Section 7.6.7 "Nail-Head Corrosion Protection." Available sizes of headed studs can be found in Appendix A and in references provided by manufacturers.





From FHWA NHI 14-007/GEC No. 7:

D. Facing Design
1) Nail head load
2) Wall facing type and thickness
3) Facing materials
4) Flexural resistance
5) Facing punching shear resistance
6) Facing head stud resistance

The safety factors correspond to the potential failure modes of the nail-facing connection including the flexural and punching shear failures. Because a two-phase facing construction is used in this project, flexural and shear-punching failure modes must be evaluated separately for the temporary and the permanent facing. Additionally, for the final facing, a tensile failure of the headed studs is considered.

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BY	JMB	DATE	11/23/2021	2000 S. Colorado Blvd., Ste 2-1000
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Facing Design - Section B-B

Station 2+20 to 5+20

Provided:

Nail Size = #10

Grade = 75.00 ksi

S_H = 5.00 ft

S_V = 5.00 ft

Nail Head Force: 33 kips static; 32 kips seismic

Loads provided are from Geotech and have been converted to To (design nail tensile force).

Step 1 - Calculate the Design Nail Head Tensile Force - Provided by Geotechs (did not need to compute)

Permanent Static:

To = 33.00 kips

γ_{EV} = 1.35 (AASHTO LRFD, Table 3.4.1-2, EV for retaining walls, per FHWA)

Seismic:

To = 32.00 kips (provided by Geotech)

F_{EQ} = 0 kips (added for max height of panel above grade)

To = 32.00 kips

Y = 1.00 (AASHTO LRFD, Table 3.4.1-1, Extreme Event I)

Step 2 - Select Wall Facing Thickness:

shotcrete thickness (h) = 6.00 in (initial)

CIP Fascia thickness (h) = 9.00 in (final)

Step 3 - Select Soil Wall Materials:

Bar Nail fy = 75.00 ksi (provided by Geotech)

Initial Facing - Shotcrete

WWF fy = 60.0 ksi

f'c = 4.0 ksi Specifications 6-18.3(2), shotcrete f'c=4000 psi

WWF = 4x4 - W4.0 x W4.0

a_{hm}, a_{vm} = 0.12 in^2 /ft

Bars at Nails = 4 (2 vert, 2 horz)

Bar As = 0.20 in^2

No bars vert = 2

A_{VN} = 0.40 in^2

No bars horz = 2

A_{HN} = 0.40 in^2

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Facing Design - Section B-B

Station 2+20 to 5+20

Perm Facing - CIP Fascia

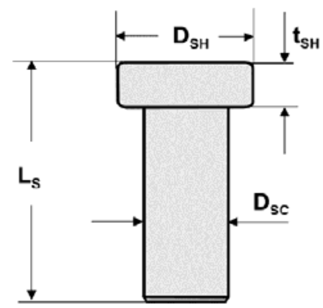
Rebar fy = 60.0 ksi
CIP bars = 5
Bar As = 0.31 in^2 / bar
spacing = 12.0 in
f'c = 4.0 ksi

TABLE A.6
HEADED-STUD DIMENSIONS

Headed-Stud Size	Nominal Length L_s mm	Nominal Length L_s in.	Head Diameter D_H mm	Head Diameter D_H in.	Shaft Diameter D_S mm	Shaft Diameter D_S in.	Head Thickness t_H mm	Head Thickness t_H in.	Head Area/ Shaft Area in.	Head Thickness/ (Head Diameter-Shaft Diameter)
$^3/_4$ x $3^{11}/_{16}$	89	15.5	31.8	1.3	19.1	0.750	9.5	0.38	2.8	0.75

Bearing Plate/Anchors

L_{BP} = 9.0 in
W_{BP} = 9.0 in
tp = 1.0 in
fy = 36.00 ksi — Grade ASTM A36
no. of studs N_H = 4 — (per BDM example)
headed stud dia D_{SC} = 0.75 in
stud shaft area As = 0.44 in^2
head size D_{SH} = 1.30 in
clr = 1.50 in
shaft L = 6.13 in
head t_{SH} = 0.375 in — (per BDM example)
Ls = shaft L + t_{SH} = 6.50 in — (max for CIP thickness)
S_{SH}, spa = 6.00 in
fy = 60.00 ksi — A307, Grade A



Step 4 - Flexural Resistance:

The facing resistance is verified in LRFD for each of the facings as follows:

$$CDR = \frac{\phi_{FF}\ R_{FF}}{\gamma\ T_o} \geq 1.0$$

Equation 6.18: Capacity-to-demand ratio for bending in facing.

Where:

ϕ_{FF} = resistance factor for bending/flexure in the facing
R_{FF} = nominal resistance for bending/flexure of facing
γ = load factor selected for verifications
T_o = maximum tensile force at soil nail head, as estimated with Eq. 5.1 (Section 5.2.1)

If the resistance is insufficient, increase the thickness of facing, amount of steel, and/or strength of steel and/or of concrete.

5.8.3c Resistance Factors for Flexure at Facing

Resistance factors for flexure resistance at the facing are back-calibrated for a load factor of γ_{EV} = 1.35 for static conditions and γ_{EV} = 1.00 for seismic loading to match ASD-based designs. These values are presented in Table 5.9.

Table 5.9: Resistance Factors for Flexure Resistance at Facing

Condition	Case	Symbol	Resistance Factor
Static	Initial and final facing	ϕ_{FF}	0.90
Seismic loading	Initial and final facing	ϕ_{FF}	0.90

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BY	JMB	DATE	11/23/2021	+1 (303) 935-6505 Fax '+1 (303) 935-6575
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Facing Design - Section B-B

Station 2+20 to 5+20

$$R_{FF}\,[\text{kip}] = 3.8 \times C_F \times f_y\,[\text{ksi}] \times F = 228 \times C_F \times F$$

Equation 6.15: Nominal bending resistance for Grade 60 steel WWM/rebar and 4,000 psi shotcrete.

Where:

$$F = \text{smaller of} \begin{cases} (a_{vn} + a_{vm})[\text{in}^2/\text{ft}] \times \left(\dfrac{S_H\ h[\text{ft}]}{S_V}\right) \\ (a_{hn} + a_{hm})[\text{in}^2/\text{ft}] \times \left(\dfrac{S_V\ h[\text{ft}]}{S_H}\right) \end{cases}$$

Equation 6.16: Definition of F.

Table 6.4: Nomenclature for Cross-Sectional Area per Unit Width of Facing Reinforcement

Direction	Location	Cross-Sectional Area of Reinforcement per Unit Width
Vertical	Nail Head (1)	a_{vn}
Vertical	Mid-Span (2)	a_{vm}
Horizontal	Nail Head (1)	a_{hn}
Horizontal	Mid-Span (2)	a_{hm}

Notes: (1) Both WWM and the rebar contribute.
(2) Only WWM contributes.

Table 6.5: Factor C_F

Facing	Facing Thickness, h_i or h_f (in.)	C_F
Initial	4	2.0
Initial	6	1.5
Initial	8	1.0
Final	All	1.0

Where:

a_{vn} = reinforcement cross-sectional area per unit width in the vertical direction at the nail head. If the yield strengths of the WWM and the rebar are different in the initial facing, this is an equivalent value.

a_{vm} = reinforcement cross-sectional area per unit width in the vertical direction at midspan

a_{hn} = equivalent reinforcement cross-sectional area per unit width in the horizontal direction at the nail head

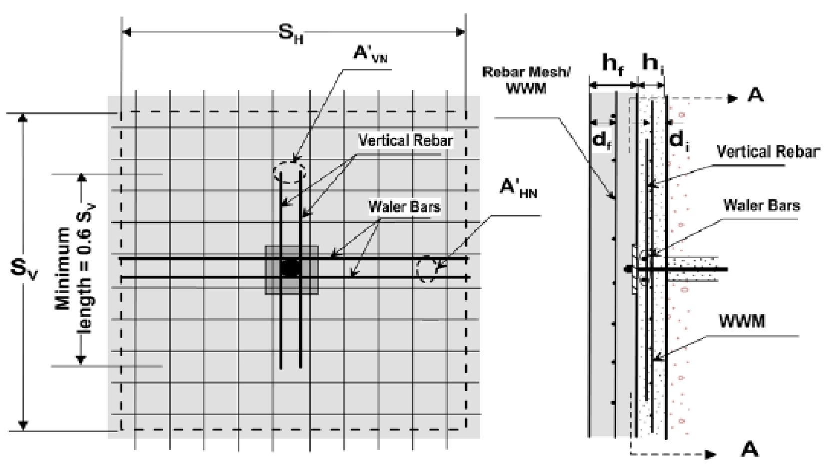
a_{hm} = reinforcement cross-sectional area per unit width in the horizontal direction at midspan

A'_{VN} = equivalent cross-sectional area at the head in the vertical direction to consider different yield strengths for the WWM and rebar. $A'_{VN} = (f_{y\_R}/f_{y\_W}) \times A_{VN}$, where f_{y_R} = rebar yield strengths and f_{y_W} = WWM yield strength

A'_{HN} = equivalent cross-sectional area at the head in the horizontal direction. $A'_{HN} = (f_{y\_R}/f_{y\_W}) \times A_{HN}$

S_H = nail horizontal spacing

S_V = nail vertical spacing



Vertical Cross Sectional Area (per unit length)   Horizontal Cross Sectional Area (per unit length)

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BY	JMB	DATE	11/23/2021		
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wood.
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Facing Design - Section B-B

Station 2+20 to 5+20

Initial Facing (Shotcrete)

a_{vm} = 0.12 in^2		a_{hm} = 0.12 in^2
$a_{vn} = a_{vm} + A_{VN}/S_H$		$a_{hn} = a_{hm} + A_{HN}/S_V$
A_{VN} = 0.40 in^2		A_{HN} = 0.40 in^2
S_H = 5.0 ft		S_V = 5.0 ft
a_{vn} = 0.20 in^2 /ft		a_{hn} = 0.20 in^2 /ft
$F = (a_{vn} + a_{vm}) * S_H/S_V*h$		$F = (a_{hn} + a_{hm}) * S_V/S_H*h$
h = 0.50 ft		h = 0.50 ft
F = smaller of: F = 0.16		**F = 0.16**

$R_{FF} = 3.8*C_F*fy*F$

C_F = 1.50 (Table 6.5, for initial, thickness = 6.00 inches)

R_{FF} = 54.72 kips

$CDR = \phi_{FF}*R_{FF}/\gamma*To$

ϕ_{FF} = 0.90 (Table 5.9)

Static: γ_{EV} = 1.35

To = 33.00 kips

Flexural Resistance **CDR = 1.11** >= 1.00 OK

Seismic γ = 1.00

To = 32.00 kips

Flexural Resistance **CDR = 1.54** >= 1.00 OK

Check reinforcement ratios:

$$\rho_{TOT}(\%) = \left[\frac{a_{vn} + a_{vm}}{12\left(\frac{h_i}{2}\right)}\right] \times 100$$

Calculate the reinforcement ratio, ρ_{ij}, as:

$$\rho_{ij} = \frac{a_{ij}}{0.5\,h}\ 100$$

Equation 6.6: Reinforcement ratio.

Where, a_{ij} = ratio of cross-sectional area of reinforcement per unit width (in "i" direction and "j" location) and h = thickness of the facing being designed, whether initial or final. The direction "i" can be "v" (for vertical) or "h" (for horizontal); the location "i" can be "n" (nail head) or "m" (mid-span between nails).

(horizontal) ρ_{TOT} = 0.89 %

(vertical) ρ_{TOT} = 0.89 %

$$\rho_{min}[\%] = 0.24\frac{\sqrt{f'_c[psi]}}{f_y[ksi]}$$

$$\rho_{max}[\%] = 0.05\frac{f'_c[psi]}{f_y[ksi]}\left(\frac{90}{90 + f_y[ksi]}\right)$$

	ρ_{min} =	0.25 %		ρ_{max} =	2.00 %	
(horizontal)	<	0.89	OK	>	0.89	OK
(vertical)	<	0.89	OK	>	0.89	OK

The additional horizontal and vertical rebar in the initial facing must be placed near the nail heads, usually within a distance h_i from the walls of the drill hole. Horizontal bars should have a length of at least $0.60S_H$; however, it is not uncommon to place waler bars with a length equal to S_H (Figure 6.5). Vertical rebar should be at least $0.60S_V$ long. Laps between adjacent bars must be in the mid-span between nails. The wire spacing in WWM typically used in soil nail walls must meet the requirements for maximum spacing of reinforcement per Section 10.6 "Distribution of Flexural Reinforcement" of ACI (2011).

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Facing Design - Section B-B

Station 2+20 to 5+20

Final Facing (CIP Fascia)

$a_{vm} = a_{vn} =$ 0.31 in^2 | $a_{hm} = a_{hn} =$ 0.31 in^2

$S_H =$ 5.0 ft | $S_V =$ 5.0 ft

$F = (a_{vn} + a_{vm}) * S_H/S_V*h$ | $F = (a_{hn} + a_{hm}) * S_V/S_H*h$

h = 0.75 ft | h = 0.75 ft

F = smaller of: F = 0.47 | **F = 0.47**

$R_{FF} = 3.8*C_F*fy*F$

$C_F =$ 1 (Table 6.5, for final)

$R_{FF} =$ 106.02 kips

$CDR = \phi_{FF}*R_{FF}/\gamma*To$

$\phi_{FF} =$ 0.90 (Table 5.9)

Static: $\gamma_{EV} =$ 1.35

To = 33.00 kips

Flexural Resistance **CDR = 2.14** >= 1.00 OK

Seismic: $\gamma =$ 1.00

To = 32.00 kips

Flexural Resistance **CDR = 2.21** >= 1.00 OK

Check reinforcement ratios:

$$\rho_{TOT}(\%) = \left[\frac{a_{vn} + a_{vm}}{12\left(\frac{h_i}{2}\right)}\right] \times 100$$

(horizontal) $\rho_{TOT} =$ 1.15 %

(vertical) $\rho_{TOT} =$ 1.15 %

Calculate the reinforcement ratio, ρ_{ij}, as:

$$\rho_{ij} = \frac{a_{ij}}{0.5\,h}\,100$$

Equation 6.6: Reinforcement ratio.

Where, a_{ij} = ratio of cross-sectional area of reinforcement per unit width (in "i" direction and "j" location) and h = thickness of the facing being designed, whether initial or final. The direction "i" can be "v" (for vertical) or "h" (for horizontal); the location "i" can be "n" (nail head) or "m" (mid-span between nails).

$$\rho_{min}[\%] = 0.24\,\frac{\sqrt{f'_c[psi]}}{f_y[ksi]}$$

$$\rho_{max}[\%] = 0.05\,\frac{f'_c[psi]}{f_y[ksi]}\left(\frac{90}{90 + f_y[ksi]}\right)$$

$\rho_{min} =$ 0.25 % | $\rho_{max} =$ 2.00 %

(horizontal) < 1.15 OK | > 1.15 OK

(vertical) < 1.15 OK | > 1.15 OK

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Facing Design - Section B-B

Station 2+20 to 5+20

Step 5 - Facing Punching Shear Resistance:

The nominal facing punching shear resistance, R_{FP}, for either situation must meet the following condition:

$$CDR = \frac{\phi_{FP}\ R_{FP}}{\gamma\ T_o} \geq 1.0$$

Equation 6.19: Capacity-to-demand ratio for punching shear resistance.

Where:

ϕ_{FP} = resistance factor for punching shear in the facing

γ = load factor selected for verification

T_o = maximum tensile force at soil nail head, as defined previously

R_{FP} is estimated as:

$$R_{FP} = C_P\ V_F$$

Equation 6.20: Nominal punching shear resistance at facing.

Where:

C_P = dimensionless factor that accounts for the contribution of the soil support under the nail head to the shear resistance

V_F = concrete punching shear basic resistance acting through the facing section

5.8.4 Punching Shear Strength Limit State

Resistance factors for punching at the facing are back-calibrated for a load factor of γ_{EV} = 1.35 for static conditions and γ_{EV} = 1.00 for seismic loading to match ASD-based designs. These values are presented in Table 5.10.

Table 5.10: Resistance Factors for Punching Shear at Facing

Condition	Case	Symbol	Resistance Factor
Static	Initial and final facing	ϕ_{FP}	0.90
Seismic loading	Initial and final facing	ϕ_{FP}	0.90

C_P can be as high as 1.15 if the soil reaction is considered. The contribution from the soil support behind the wall is conservatively assumed to be negligible; therefore, C_P = 1.0. Tl

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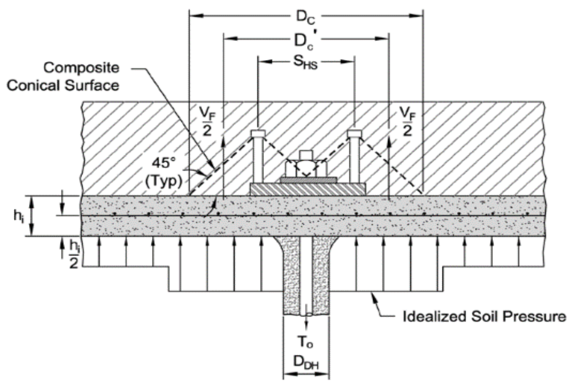
Facing Design - Section B-B

Station 2+20 to 5+20

$$V_F\,[\text{kip}] = 0.58\sqrt{f'_c\ [\text{psi}]}\ \pi\ D'_c\,[\text{ft}]\,h_c\,[\text{ft}]$$

Equation 6.21: Nominal punching shear resistance through facing.



(b) Headed Stud Connection

Initial Facing (Shotcrete)

$D'c = L_{BP} + h_i$
$L_{BP} =$ 9.00 in
$hc = h_i =$ 6.00 in
$D'c =$ 15.00 in
$V_F =$ 71.99 kips

$C_P =$ 1.00 (assume negligible)
$R_{FP} = C_P*V_F =$ 71.99 kips

$CDR = \phi_{FP}*R_{FP}/\gamma*To$

$\phi_{FP} =$ 0.90 (Table 5.10)

Static: $\gamma_{EV} =$ 1.35
To = 33.00 kips

Punching Shear **CDR = 1.45** >= 1.00 OK

Seismic: $\gamma =$ 1.00
To = 32.00 kips

Punching Shear **CDR = 1.50** >= 1.00 OK

Final Facing (CIP Fascia)

$D'c =$ smaller $S_{SH}+h_c$ or $2*h_c$
$hc = L_S + tp - t_{SH}$
$L_S =$ 6.50 in
$tp =$ 1.0 in
$t_{SH} =$ 0.375 in
$hc =$ 7.125 in
$S_{SH} =$ 6.00 in
$S_{SH}+h_c =$ 13.125 in
$2*h_c =$ 14.25 in
D'c (min) = 13.125 in
$V_F =$ 74.80 kips

$C_P =$ 1.00 (assume negligible)
$R_{FP} = C_P*V_F =$ 74.80 kips

$CDR = \phi_{FP}*R_{FP}/\gamma*To$

$\phi_{FP} =$ 0.90 (Table 5.9)

Static: $\gamma_{EV} =$ 1.35
To = 33.00 kips

Punching Shear **CDR = 1.51** >= 1.00 OK

Seismic: $\gamma =$ 1.00
To = 32.00 kips

Punching Shear **CDR = 1.56** >= 1.00 OK

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Facing Design - Section B-B

Station 2+20 to 5+20

Step 6 - Facing Head Stud Resistance:

$$CDR = \frac{\phi_{FH}\ R_{FH}}{\gamma\ T_o} \geq 1.0$$

Equation 6.27: Capacity-to-demand ratio for a headed stud.

Where:

ϕ_{FH} = resistance factor for headed stud tensile resistance

R_{FH} = nominal tensile resistance of headed studs in final facings

γ = load factor selected for verification

T_o = as defined earlier

$$R_{FH} = N_H\ A_S\ f_{y\text{-}hs}$$

Equation 6.26: Headed stud resistance at facing.

Where:

N_H = number of headed studs in the connection (usually 4)

A_S = cross-sectional area of the headed stud shaft

$f_{y\text{-}hs}$ = tensile yield strength of headed stud

5.8.5 Headed Stud in Tension in Final Facing

Resistance factors for headed stud in tension punching in the final facing are back-calibrated for a load factor of γ_{EV} = 1.35 for static conditions and γ_{EV} = 1.00 for seismic loading to match ASD-based designs. These values are presented in Table 5.11.

Table 5.11: Resistance Factors for Headed Stud in Tension in Final Facings

Condition	Case	Symbol	Resistance Factor
Static	A307 Steel Bolt (1)	ϕ_{FH}	0.70
Static	A325 Steel Bolt	ϕ_{FH}	0.80
Seismic loading	A307 Steel Bolt (1)	ϕ_{FH}	0.65
Seismic loading	A325 Steel Bolt	ϕ_{FH}	0.75

Note: (1) This is equivalent to AWS D1.1 Type B studs, with f_y = 60 ksi.

N_H = 4
As = 0.44 in^2
fy = 60.00 ksi
R_{FH} = 105.98 kips

CDR = ϕ_{FP}*R_{FP}/γ*To

ϕ_{FP} = 0.70 (Table 5.11)

Static: γ_{EV} = 1.35
To = 33.00 kips
Stud Resistance **CDR = 1.67** **>= 1.00 OK**

Seismic: γ = 1.00
To = 32.00 kips
Stud Resistance **CDR = 1.72** **>= 1.00 OK**

Summary

Shotcrete Layer:
thickness = 6.00 in
main reinf = 4x4 - W4.0 x W4.0 Horz bar L = 0.6Sh = 3.00 ft
bar size at nails = 4 (2 vert and 2 horz) Vert bar L = 0.6Sv = 3.00 ft

CIP Fascia
thickness = 9.00 in
bar size 5 spaced at 12.0 in

Bearing Plate
square - size = 9.0 x 9.0 x 1.0 in thick
4 anchors spaced at = 6.00 in
anchor L = 6.50 in
anchor D = 0.75 in

GOVERNS FOR SECTIONS - SEE TOP ROW MAX FOR FINAL THICKNESS OF SHOTCRETE AND CIP CONCRETE

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Facing Design - Section C-C

North End of Wall

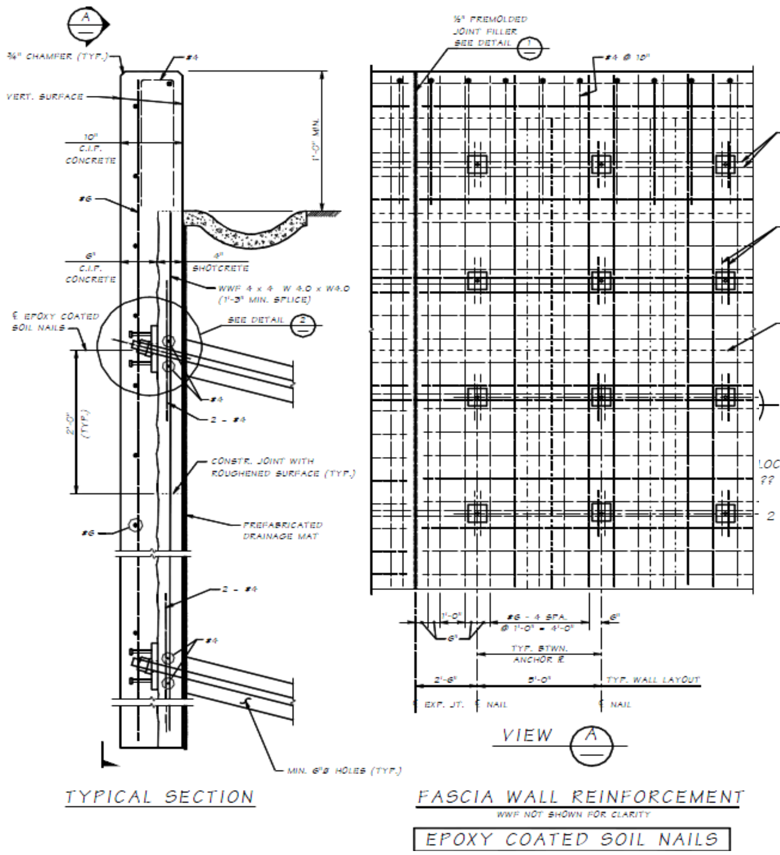
Station 5+20 to 6+06 (end)

L = 20'

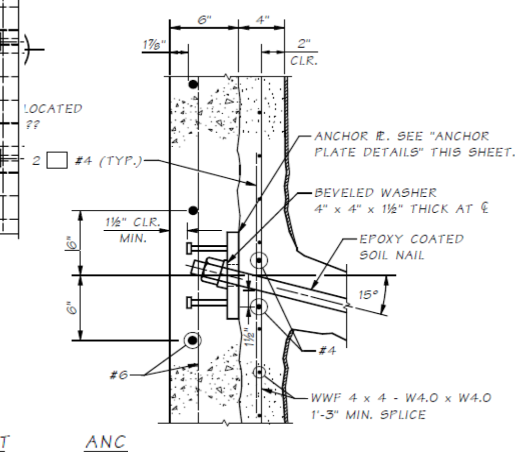
DTL = 1,700 plf for Top Row; 2,500 plf for Remaining Rows

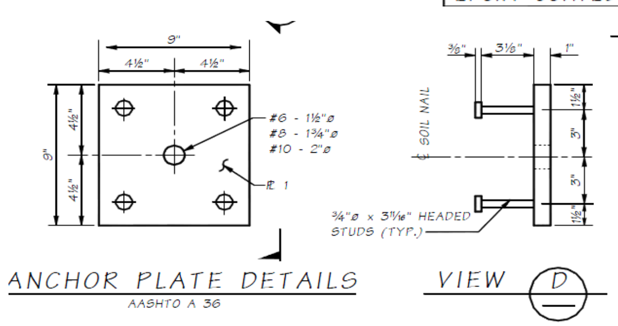
Nail Head Force: 27 kips static; 27 kips seismic

Will use BDM Standard Sheet for dimensions and confirm with analysis along with spacing and nail information provided by Geotech:

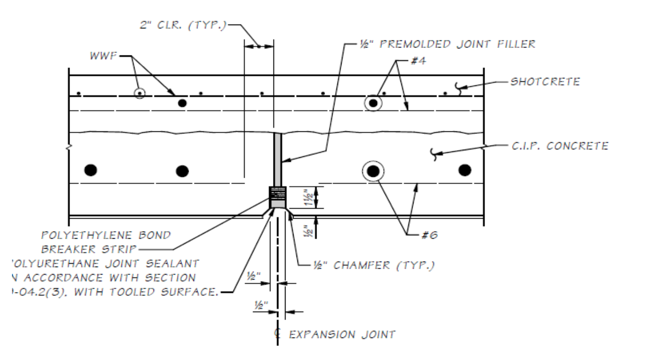


Per FHWA - need min 2" CLR for studs for corrosion protection - all walls will have additional ~1-1.5" of formliner - therefore will keep 1.5" clr for calculations.





Note that L_S and h_f must be selected such that the cover of final shotcrete or concrete over the stud is at least 2 in. according to Section 7.6.7 "Nail-Head Corrosion Protection." Available sizes of headed studs can be found in Appendix A and in references provided by manufacturers.



From FHWA NHI 14-007/GEC No. 7:

D. Facing Design
1) Nail head load
2) Wall facing type and thickness
3) Facing materials
4) Flexural resistance
5) Facing punching shear resistance
6) Facing head stud resistance

The safety factors correspond to the potential failure modes of the nail-facing connection including the flexural and punching shear failures. Because a two-phase facing construction is used in this project, flexural and shear-punching failure modes must be evaluated separately for the temporary and the permanent facing. Additionally, for the final facing, a tensile failure of the headed studs is considered.

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Facing Design - Section C-C

Station 5+20 to 6+06 (end)

Provided:

Nail Size = #8
Grade = 75.00 ksi
S_H = 6.00 ft
S_V = 5.00 ft

Nail Head Force: 27 kips static; 27 kips seismic

Loads provided are from Geotech and have been converted to To (design nail tensile force).

Step 1 - Calculate the Design Nail Head Tensile Force - Provided by Geotechs (did not need to compute)

Permanent Static:

To = 27.00 kips
γ_{EV} = 1.35 (AASHTO LRFD, Table 3.4.1-2, EV for retaining walls, per FHWA)

Seismic:

To = 27.00 kips
Y = 1.00 (AASHTO LRFD, Table 3.4.1-1, Extreme Event I)

Step 2 - Select Wall Facing Thickness:

shotcrete thickness (h) = 6.00 in (initial)
CIP Fascia thickness (h) = 9.00 in (final)

Step 3 - Select Soil Wall Materials:

Bar Nail fy = 75.00 ksi (provided by Geotech)

Initial Facing - Shotcrete

WWF fy = 60.0 ksi
f'c = 4.0 ksi Specifications 6-18.3(2), shotcrete f'c=4000 psi
WWF = 4x4 - W4.0 x W4.0
a_{hm}, a_{vm} = 0.12 in^2 /ft
Bars at Nails = 4 (2 vert, 2 horz)
Bar As = 0.20 in^2
No bars vert = 2
A_{VN} = 0.40 in^2
No bars horz = 2
A_{HN} = 0.40 in^2

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Facing Design - Section C-C

Station 5+20 to 6+06 (end)

Perm Facing - CIP Fascia

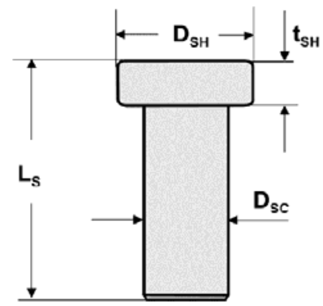
Rebar fy =	60.0	ksi
CIP bars =	5	
Bar As =	0.31	in^2 / bar
spacing =	12.0	in
f'c =	4.0	ksi

TABLE A.6
HEADED-STUD DIMENSIONS

Headed-Stud Size	Nominal Length L_s		Head Diameter D_H		Shaft Diameter D_s		Head Thickness t_H		Head Area/ Shaft Area	Head Thickness/ (Head Diameter-Shaft Diameter)
	mm	in.	mm	in.	mm	in.	mm	in.	in.	
$^{3}/_{4}$ x 3 $^{11}/_{16}$	89	15.5	31.8	1.3	19.1	0.750	9.5	0.38	2.8	0.75

Bearing Plate/Anchors

L_{BP} =	9.0	in	
W_{BP} =	9.0	in	
tp =	1.0	in	
fy =	36.00	ksi	Grade ASTM A36
no. of studs N_H =	4		(per BDM example)
headed stud dia D_{SC} =	0.75	in	
stud shaft area As =	0.44	in^2	
head size D_{SH} =	1.30	in	
clr =	1.50	in	
shaft L =	6.13	in	
head t_{SH} =	0.375	in	(per BDM example)
Ls = shaft L + t_{SH} =	6.50	in	(max for CIP thickness)
S_{SH}, spa =	6.00	in	
fy =	60.00	ksi	A307, Grade A



Step 4 - Flexural Resistance:

The facing resistance is verified in LRFD for each of the facings as follows:

$$CDR = \frac{\phi_{FF}\ R_{FF}}{\gamma\ T_o} \geq 1.0$$

Equation 6.18: Capacity-to-demand ratio for bending in facing.

Where:

ϕ_{FF} = resistance factor for bending/flexure in the facing

R_{FF} = nominal resistance for bending/flexure of facing

γ = load factor selected for verifications

T_o = maximum tensile force at soil nail head, as estimated with Eq. 5.1 (Section 5.2.1)

If the resistance is insufficient, increase the thickness of facing, amount of steel, and/or strength of steel and/or of concrete.

5.8.3c Resistance Factors for Flexure at Facing

Resistance factors for flexure resistance at the facing are back-calibrated for a load factor of γ_{EV} = 1.35 for static conditions and γ_{EV} = 1.00 for seismic loading to match ASD-based designs. These values are presented in Table 5.9.

Table 5.9: Resistance Factors for Flexure Resistance at Facing

Condition	Case	Symbol	Resistance Factor
Static	Initial and final facing	ϕ_{FF}	0.90
Seismic loading	Initial and final facing	ϕ_{FF}	0.90

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Facing Design - Section C-C

Station 5+20 to 6+06 (end)

$$R_{FF}[\text{kip}] = 3.8 \times C_F \times f_y[\text{ksi}] \times F = 228 \times C_F \times F$$

Equation 6.15: Nominal bending resistance for Grade 60 steel WWM/rebar and 4,000 psi shotcrete.

Where:

$$F = \text{smaller of} \begin{cases} (a_{vn} + a_{vm})[\text{in}^2/\text{ft}] \times \left(\frac{S_H\ h[\text{ft}]}{S_V}\right) \\ (a_{hn} + a_{hm})[\text{in}^2/\text{ft}] \times \left(\frac{S_V\ h[\text{ft}]}{S_H}\right) \end{cases}$$

Equation 6.16: Definition of F.

Table 6.4: Nomenclature for Cross-Sectional Area per Unit Width of Facing Reinforcement

Direction	Location	Cross-Sectional Area of Reinforcement per Unit Width
Vertical	Nail Head (1)	a_{vn}
Vertical	Mid-Span (2)	a_{vm}
Horizontal	Nail Head (1)	a_{hn}
Horizontal	Mid-Span (2)	a_{hm}

Notes: (1) Both WWM and the rebar contribute.
(2) Only WWM contributes.

Table 6.5: Factor C_F

Facing	Facing Thickness, h_i or h_f (in.)	C_F
Initial	4	2.0
Initial	6	1.5
Initial	8	1.0
Final	All	1.0

Where:

a_{vn} = reinforcement cross-sectional area per unit width in the vertical direction at the nail head. If the yield strengths of the WWM and the rebar are different in the initial facing, this is an equivalent value.

a_{vm} = reinforcement cross-sectional area per unit width in the vertical direction at midspan

a_{hn} = equivalent reinforcement cross-sectional area per unit width in the horizontal direction at the nail head

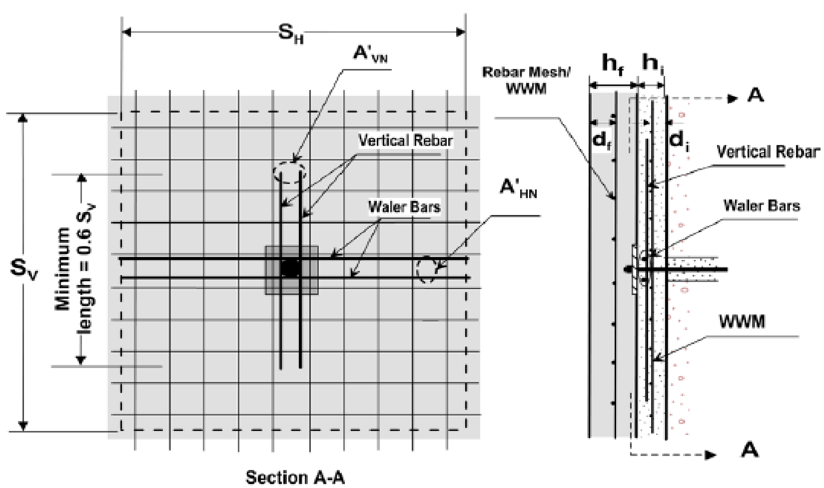
a_{hm} = reinforcement cross-sectional area per unit width in the horizontal direction at midspan

A'_{VN} = equivalent cross-sectional area at the head in the vertical direction to consider different yield strengths for the WWM and rebar. $A'_{VN} = (f_{y\_R}/f_{y\_W}) \times A_{VN}$, where f_{y_R} = rebar yield strengths and f_{y_W} = WWM yield strength

A'_{HN} = equivalent cross-sectional area at the head in the horizontal direction. $A'_{HN} = (f_{y\_R}/f_{y\_W}) \times A_{HN}$

S_H = nail horizontal spacing

S_V = nail vertical spacing



Vertical Cross Sectional Area (per unit length)   Horizontal Cross Sectional Area (per unit length)

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Facing Design - Section C-C

Station 5+20 to 6+06 (end)

Initial Facing (Shotcrete)

a_{vm} =	0.12	in^2	a_{hm} =	0.12	in^2
$a_{vn} = a_{vm} + A_{VN}/S_H$			$a_{hn} = a_{hm} + A_{HN}/S_V$		
A_{VN} =	0.40	in^2	A_{HN} =	0.40	in^2
S_H =	6.0	ft	S_V =	5.0	ft
a_{vn} =	0.19	in^2 /ft	a_{hn} =	0.20	in^2 /ft
$F = (a_{vn} + a_{vm}) * S_H/S_V*h$			$F = (a_{hn} + a_{hm}) * S_V/S_H*h$		
h =	0.50	ft	h =	0.50	ft
F = smaller of: F =	0.18		**F =**	**0.13**	

$R_{FF} = 3.8*C_F*fy*F$

C_F = 1.50 (Table 6.5, for initial, thickness = 6.00 inches)

R_{FF} = 45.60 kips

$CDR = \phi_{FF}*R_{FF}/\gamma*To$

ϕ_{FF} = 0.90 (Table 5.9)

Static:

γ_{EV} = 1.35

To = 27.00 kips

Flexural Resistance **CDR = 1.13** **>= 1.00 OK**

Seismic

γ = 1.00

To = 27.00 kips

Flexural Resistance **CDR = 1.52** **>= 1.00 OK**

Check reinforcement ratios:

$$\rho_{TOT}\,(\%) = \left[\frac{a_{vn} + a_{vm}}{12\left(\frac{h_i}{2}\right)}\right] \times 100$$

Calculate the reinforcement ratio, ρ_{ij}, as:

$$\rho_{ij} = \frac{a_{ij}}{0.5\,h}\ 100$$

Equation 6.6: Reinforcement ratio.

Where, a_{ij} = ratio of cross-sectional area of reinforcement per unit width (in "i" direction and "j" location) and h = thickness of the facing being designed, whether initial or final. The direction "i" can be "v" (for vertical) or "h" (for horizontal); the location "i" can be "n" (nail head) or "m" (mid-span between nails).

(horizontal) ρ_{TOT} = 0.89 %

(vertical) ρ_{TOT} = 0.85 %

$$\rho_{min}[\%] = 0.24\,\frac{\sqrt{f'_c[psi]}}{f_y[ksi]}$$

$$\rho_{max}[\%] = 0.05\,\frac{f'_c[psi]}{f_y[ksi]}\left(\frac{90}{90+f_y[ksi]}\right)$$

	ρ_{min} =	0.25 %		ρ_{max} =	2.00 %	
(horizontal)	<	0.89	OK	>	0.89	OK
(vertical)	<	0.85	OK	>	0.85	OK

The additional horizontal and vertical rebar in the initial facing must be placed near the nail heads, usually within a distance h_i from the walls of the drill hole. Horizontal bars should have a length of at least $0.60S_H$; however, it is not uncommon to place waler bars with a length equal to S_H (Figure 6.5). Vertical rebar should be at least $0.60S_V$ long. Laps between adjacent bars must be in the mid-span between nails. The wire spacing in WWM typically used in soil nail walls must meet the requirements for maximum spacing of reinforcement per Section 10.6 "Distribution of Flexural Reinforcement" of ACI (2011).

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Facing Design - Section C-C

Station 5+20 to 6+06 (end)

Final Facing (CIP Fascia)

$a_{vm} = a_{vn} =$ 0.31 in^2		$a_{hm} = a_{hn} =$ 0.31 in^2
$S_H =$ 6.0 ft		$S_V =$ 5.0 ft
$F = (a_{vn} + a_{vm}) * S_H/S_V*h$		$F = (a_{hn} + a_{hm}) * S_V/S_H*h$
h = 0.75 ft		h = 0.75 ft
F = smaller of:	F = 0.56	**F = 0.39**

$R_{FF} = 3.8*C_F*fy*F$

$C_F =$ 1 (Table 6.5, for final)

$R_{FF} =$ 88.35 kips

$CDR = \phi_{FF}*R_{FF}/\gamma*To$

$\phi_{FF} =$ 0.90 (Table 5.9)

Static: $\gamma_{EV} =$ 1.35

To = 27.00 kips

Flexural Resistance **CDR = 2.18** **>= 1.00 OK**

Seismic: $\gamma =$ 1.00

To = 27.00 kips

Flexural Resistance **CDR = 2.18** **>= 1.00 OK**

Check reinforcement ratios:

$$\rho_{TOT}(\%) = \left[\frac{a_{vn} + a_{vm}}{12\left(\frac{h_i}{2}\right)}\right] \times 100$$

(horizontal) $\rho_{TOT} =$ 1.15 %

(vertical) $\rho_{TOT} =$ 1.15 %

Calculate the reinforcement ratio, ρ_{ij}, as:

$$\rho_{ij} = \frac{a_{ij}}{0.5\,h}\,100$$

Equation 6.6: Reinforcement ratio.

Where, a_{ij} = ratio of cross-sectional area of reinforcement per unit width (in "i" direction and "j" location) and h = thickness of the facing being designed, whether initial or final. The direction "i" can be "v" (for vertical) or "h" (for horizontal); the location "i" can be "n" (nail head) or "m" (mid-span between nails).

$$\rho_{min}[\%] = 0.24\,\frac{\sqrt{f'_c[psi]}}{f_y[ksi]}$$

$$\rho_{max}[\%] = 0.05\,\frac{f'_c[psi]}{f_y[ksi]}\left(\frac{90}{90+f_y[ksi]}\right)$$

	$\rho_{min} =$ 0.25 %	$\rho_{max} =$ 2.00 %
(horizontal)	**< 1.15 OK**	**> 1.15 OK**
(vertical)	**< 1.15 OK**	**> 1.15 OK**

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Facing Design - Section C-C

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Step 5 - Facing Punching Shear Resistance:

The nominal facing punching shear resistance, R_{FP}, for either situation must meet the following condition:

$$CDR = \frac{\phi_{FP}\ R_{FP}}{\gamma\ T_o} \geq 1.0$$

Equation 6.19: Capacity-to-demand ratio for punching shear resistance.

Where:

ϕ_{FP} = resistance factor for punching shear in the facing

γ = load factor selected for verification

T_o = maximum tensile force at soil nail head, as defined previously

R_{FP} is estimated as:

$$R_{FP} = C_P\ V_F$$

Equation 6.20: Nominal punching shear resistance at facing.

Where:

C_P = dimensionless factor that accounts for the contribution of the soil support under the nail head to the shear resistance

V_F = concrete punching shear basic resistance acting through the facing section

5.8.4 Punching Shear Strength Limit State

Resistance factors for punching at the facing are back-calibrated for a load factor of $\gamma_{EV} = 1.35$ for static conditions and $\gamma_{EV} = 1.00$ for seismic loading to match ASD-based designs. These values are presented in Table 5.10.

Table 5.10: Resistance Factors for Punching Shear at Facing

Condition	Case	Symbol	Resistance Factor
Static	Initial and final facing	ϕ_{FP}	0.90
Seismic loading	Initial and final facing	ϕ_{FP}	0.90

C_P can be as high as 1.15 if the soil reaction is considered. The contribution from the soil support behind the wall is conservatively assumed to be negligible; therefore, $C_P = 1.0$. Tl

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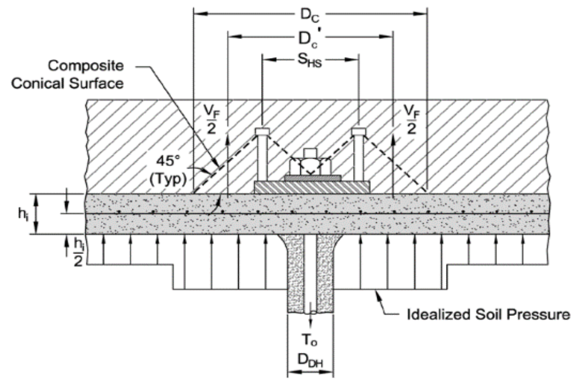
$$V_F\,[\text{kip}] = 0.58\sqrt{f'_c\ [\text{psi}]}\ \pi\ D'_c\,[\text{ft}]\,h_c\,[\text{ft}]$$

Equation 6.21: Nominal punching shear resistance through facing.



(b) Headed Stud Connection

Initial Facing (Shotcrete)

$D'c = L_{BP} + h_i$

$L_{BP} =$ 9.00 in
$hc = h_i =$ 6.00 in
$D'c =$ 15.00 in
$V_F =$ 71.99 kips

$C_P =$ 1.00 (assume negligible)
$R_{FP} = C_P{*}V_F =$ 71.99 kips

$CDR = \phi_{FP}{*}R_{FP}/\gamma{*}To$

$\phi_{FP} =$ 0.90 (Table 5.10)

Static: $\gamma_{EV} =$ 1.35
To = 27.00 kips
Punching Shear **CDR = 1.78** >= 1.00 **OK**

Seismic: $\gamma =$ 1.00
To = 27.00 kips
Punching Shear **CDR = 1.78** >= 1.00 **OK**

Final Facing (CIP Fascia)

$D'c =$ smaller $S_{SH}+h_c$ or $2{*}h_c$

$hc = L_S + tp - t_{SH}$

$L_S =$ 6.50 in
tp = 1.0 in
$t_{SH} =$ 0.375 in
hc = 7.125 in
$S_{SH} =$ 6.00 in
$S_{SH}+h_c =$ 13.125 in
$2{*}h_c =$ 14.25 in
D'c (min) = 13.125 in
$V_F =$ 74.80 kips

$C_P =$ 1.00 (assume negligible)
$R_{FP} = C_P{*}V_F =$ 74.80 kips

$CDR = \phi_{FP}{*}R_{FP}/\gamma{*}To$

$\phi_{FP} =$ 0.90 (Table 5.9)

Static: $\gamma_{EV} =$ 1.35
To = 27.00 kips
Punching Shear **CDR = 1.85** >= 1.00 **OK**

Seismic: $\gamma =$ 1.00
To = 27.00 kips
Punching Shear **CDR = 1.85** >= 1.00 **OK**

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Facing Design - Section C-C

Station 5+20 to 6+06 (end)

Step 6 - Facing Head Stud Resistance:

$$CDR = \frac{\phi_{FH}\ R_{FH}}{\gamma\ T_o} \geq 1.0$$

Equation 6.27: Capacity-to-demand ratio for a headed stud.

Where:

ϕ_{FH} = resistance factor for headed stud tensile resistance
R_{FH} = nominal tensile resistance of headed studs in final facings
γ = load factor selected for verification
T_o = as defined earlier

$$R_{FH} = N_H\ A_S\ f_{y\text{-}hs}$$

Equation 6.26: Headed stud resistance at facing.

Where:

N_H = number of headed studs in the connection (usually 4)
A_S = cross-sectional area of the headed stud shaft
$f_{y\text{-}hs}$ = tensile yield strength of headed stud

5.8.5 Headed Stud in Tension in Final Facing

Resistance factors for headed stud in tension punching in the final facing are back-calibrated for a load factor of γ_{EV} = 1.35 for static conditions and γ_{EV} = 1.00 for seismic loading to match ASD-based designs. These values are presented in Table 5.11.

Table 5.11: Resistance Factors for Headed Stud in Tension in Final Facings

Condition	Case	Symbol	Resistance Factor
Static	A307 Steel Bolt (1)	ϕ_{FH}	0.70
Static	A325 Steel Bolt	ϕ_{FH}	0.80
Seismic loading	A307 Steel Bolt (1)	ϕ_{FH}	0.65
Seismic loading	A325 Steel Bolt	ϕ_{FH}	0.75

Note: (1) This is equivalent to AWS D1.1 Type B studs, with f_y = 60 ksi.

N_H = 4
As = 0.44 in^2
fy = 60.00 ksi
R_{FH} = 105.98 kips

CDR = ϕ_{FP}*R_{FP}/γ*To

ϕ_{FP} = 0.70 (Table 5.11)

Static:
γ_{EV} = 1.35
To = 27.00 kips
Stud Resistance **CDR = 2.04** **>= 1.00 OK**

Seismic:
γ = 1.00
To = 27.00 kips
Stud Resistance **CDR = 2.04** **>= 1.00 OK**

Summary

Shotcrete Layer:
thickness = 6.00 in
main reinf = 4x4 - W4.0 x W4.0 — Horz bar L = 0.6Sh = 3.60 ft
bar size at nails = 4 (2 vert and 2 horz) — Vert bar L = 0.6Sv = 3.00 ft

CIP Fascia
thickness = 9.00 in
bar size 5 spaced at 12.0 in

Bearing Plate
square - size = 9.0 x 9.0 x 1.0 in thick
4 anchors spaced at = 6.00 in
anchor L = 6.50 in
anchor D = 0.75 in

DOES NOT GOVERN

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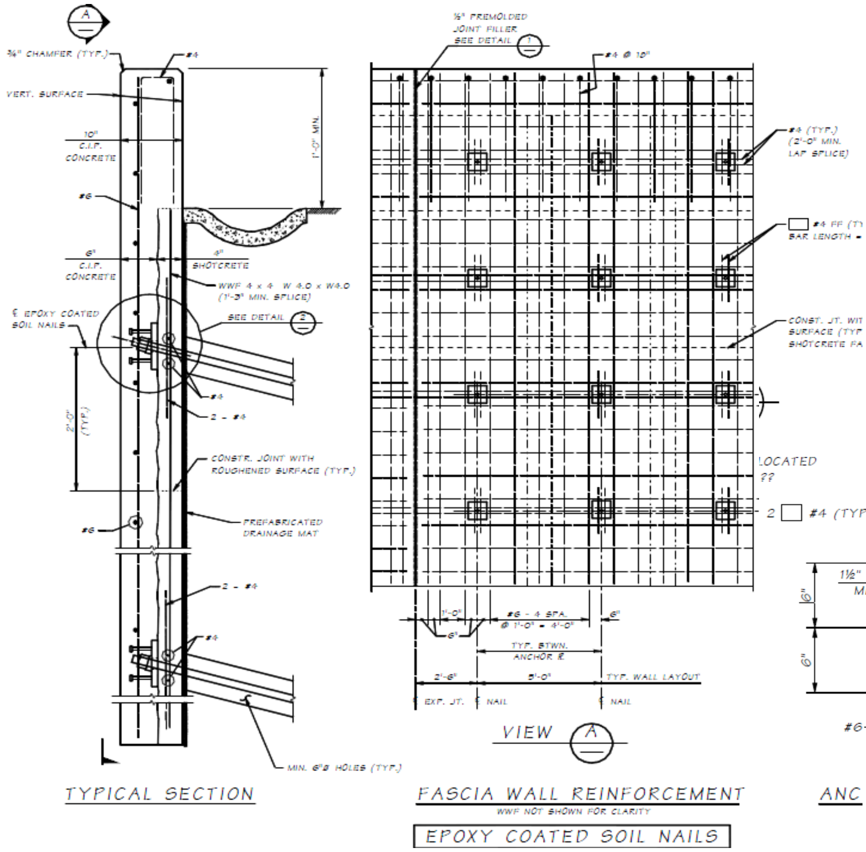
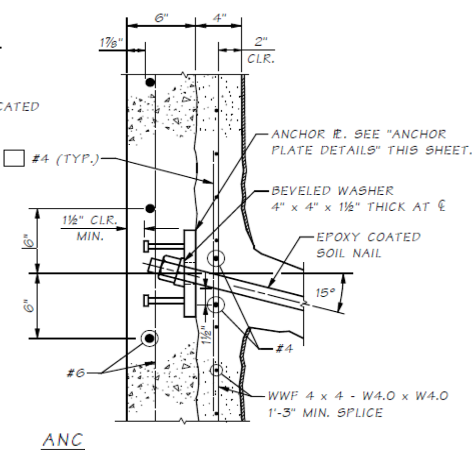
Facing Design - Top Row of Nails - Governing Condition

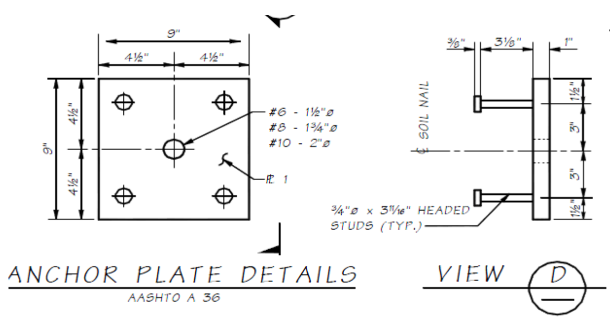
This design check is for the top row of nails to account for the facing panel extensions above grade. The governing (highest) panel is at STA 2+38 and will be ~9ft above grade to top of concrete panel. The capacity of this section was checked similar to a noise wall (fixed at the first nail) for a railing load, wind load and seismic load per AASHTO to confirm the facing thickness and required reinforcement. The governing load condition was computed to be the extreme (seismic) condition. The additional force acting on the top row of nails is added to the provided nail forces to check facing design and connection details.

Will use BDM Standard Sheet for dimensions and confirm with analysis along with spacing and nail information provided by Geotech:

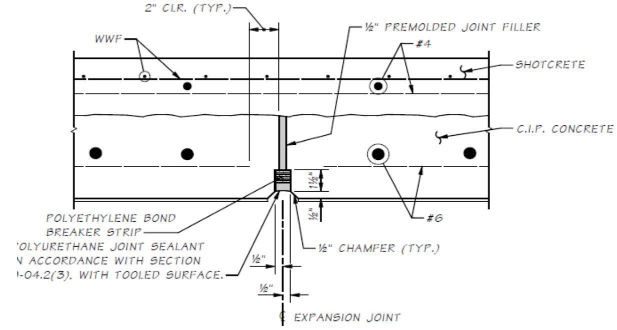


Per FHWA - need min 2" CLR for studs for corrosion protection - all walls will have additional ~1-1.5" of formliner - therefore will keep 1.5" clr for calculations.





Note that L_S and h_r must be selected such that the cover of final shotcrete or concrete over the stud is at least 2 in. according to Section 7.6.7 "Nail-Head Corrosion Protection." Available sizes of headed studs can be found in Appendix A and in references provided by manufacturers.



From FHWA NHI 14-007/GEC No. 7:

D. Facing Design
1) Nail head load
2) Wall facing type and thickness
3) Facing materials
4) Flexural resistance
5) Facing punching shear resistance
6) Facing head stud resistance

The safety factors correspond to the potential failure modes of the nail-facing connection including the flexural and punching shear failures. Because a two-phase facing construction is used in this project, flexural and shear-punching failure modes must be evaluated separately for the temporary and the permanent facing. Additionally, for the final facing, a tensile failure of the headed studs is considered.

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Facing Design - Top Row of Nails - Governing Condition

Provided:

Location = Section B-B - where max panel is located and also max load provided by geotch

Nail Size = #10

Grade = 75.00 ksi

S_H = 5.00 ft

S_V = 5.00 ft

Loads provided are from Geotech and have been converted to To (design nail tensile force).

Step 1 - Calculate the Design Nail Head Tensile Force - Provided by Geotechs (did not need to compute)

Seismic:

Table: Nails & max mobilized head forces

Name	Nail Section	α deg	x (ft)	El. (ft)	Lfix (ft)	Lfree (ft)	Space (ft)	Fhead (k/ft)	Fhead (k)
-									
N0	0: N1	15	0	175	25	0	5	6.2515	31.26

Design load at the top nail was provided for Max Section B-B (at STA 3+80) - which represents the area where this max section is located - conservative, as wall height at step location is lower.

To = 31.26 kips (Provided by Geotech)

F_{EQ} = 15.71 kips (added for max height of panel above grade - see separate seismic calcs)

To = 46.97 kips

Y = 1.00 (AASHTO LRFD, Table 3.4.1-1, Extreme Event I)

Step 2 - Select Wall Facing Thickness:

shotcrete thickness (h) = 6.00 in (initial)

CIP Fascia thickness (h) = 9.00 in (final)

Step 3 - Select Soil Wall Materials:

Bar Nail fy = 75.00 ksi (provided by Geotech)

Initial Facing - Shotcrete

WWF fy = 60.0 ksi

f'c = 4.0 ksi Specifications 6-18.3(2), shotcrete f'c=4000 psi

WWF = 4x4 - W4.0 x W4.0

a_{hm}, a_{vm} = 0.12 in^2 /ft

Bars at Nails = 4 (2 vert, 2 horz)

Bar As = 0.20 in^2

No bars vert = 2

A_{VN} = 0.40 in^2

No bars horz = 2

A_{HN} = 0.40 in^2

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Facing Design - Top Row of Nails - Governing Condition

Perm Facing - CIP Fascia

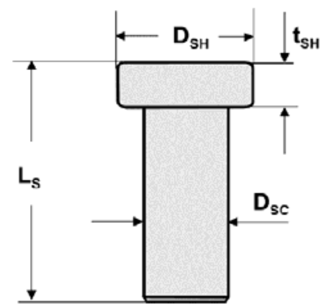
Rebar fy =	60.0	ksi
CIP bars =	5	
Bar As =	0.31	in^2 / bar
spacing =	12.0	in
f'c =	4.0	ksi

TABLE A.6
HEADED-STUD DIMENSIONS

Headed-Stud Size	Nominal Length L_s		Head Diameter D_H		Shaft Diameter D_S		Head Thickness t_H		Head Area/ Shaft Area	Head Thickness/ (Head Diameter-Shaft Diameter)
	mm	in.	mm	in.	mm	in.	mm	in.	in.	
$^3/_4$ x $3^{11}/_{16}$	89	15.5	31.8	1.3	19.1	0.750	9.5	0.38	2.8	0.75

Bearing Plate/Anchors

L_{BP} =	9.0	in	
W_{BP} =	9.0	in	
tp =	1.0	in	
fy =	36.00	ksi	Grade ASTM A36
no. of studs N_H =	4		(per BDM example)
headed stud dia D_{SC} =	0.75	in	
stud shaft area As =	0.44	in^2	
head size D_{SH} =	1.30	in	
clr =	1.50	in	
shaft L =	6.13	in	
head t_{SH} =	0.375	in	(per BDM example)
Ls = shaft L + t_{SH} =	6.50	in	(max for CIP thickness)
S_{SH}, spa =	6.00	in	
fy =	60.00	ksi	A307, Grade A



Step 4 - Flexural Resistance:

The facing resistance is verified in LRFD for each of the facings as follows:

$$CDR = \frac{\phi_{FF} \ R_{FF}}{\gamma \ T_o} \geq 1.0$$

Equation 6.18: Capacity-to-demand ratio for bending in facing.

Where:

ϕ_{FF} = resistance factor for bending/flexure in the facing

R_{FF} = nominal resistance for bending/flexure of facing

γ = load factor selected for verifications

T_o = maximum tensile force at soil nail head, as estimated with Eq. 5.1 (Section 5.2.1)

If the resistance is insufficient, increase the thickness of facing, amount of steel, and/or strength of steel and/or of concrete.

5.8.3c Resistance Factors for Flexure at Facing

Resistance factors for flexure resistance at the facing are back-calibrated for a load factor of γ_{EV} = 1.35 for static conditions and γ_{EV} = 1.00 for seismic loading to match ASD-based designs. These values are presented in Table 5.9.

Table 5.9: Resistance Factors for Flexure Resistance at Facing

Condition	Case	Symbol	Resistance Factor
Static	Initial and final facing	ϕ_{FF}	0.90
Seismic loading	Initial and final facing	ϕ_{FF}	0.90

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Facing Design - Top Row of Nails - Governing Condition

$$R_{FF}\,[\text{kip}] = 3.8 \times C_F \times f_y\,[\text{ksi}] \times F = 228 \times C_F \times F$$

Equation 6.15: Nominal bending resistance for Grade 60 steel WWM/rebar and 4,000 psi shotcrete.

Table 6.4: Nomenclature for Cross-Sectional Area per Unit Width of Facing Reinforcement

Direction	Location	Cross-Sectional Area of Reinforcement per Unit Width
Vertical	Nail Head (1)	a_{vn}
Vertical	Mid-Span (2)	a_{vm}
Horizontal	Nail Head (1)	a_{hn}
Horizontal	Mid-Span (2)	a_{hm}

Notes: (1) Both WWM and the rebar contribute.
(2) Only WWM contributes.

Where:

a_{vn} = reinforcement cross-sectional area per unit width in the vertical direction at the nail head. If the yield strengths of the WWM and the rebar are different in the initial facing, this is an equivalent value.

a_{vm} = reinforcement cross-sectional area per unit width in the vertical direction at midspan

a_{hn} = equivalent reinforcement cross-sectional area per unit width in the horizontal direction at the nail head

a_{hm} = reinforcement cross-sectional area per unit width in the horizontal direction at midspan

A'_{VN} = equivalent cross-sectional area at the head in the vertical direction to consider different yield strengths for the WWM and rebar. $A'_{VN} = (f_{y\_R}/f_{y\_W}) \times A_{VN}$, where f_{y_R} = rebar yield strengths and f_{y_W} = WWM yield strength

A'_{HN} = equivalent cross-sectional area at the head in the horizontal direction. $A'_{HN} = (f_{y\_R}/f_{y\_W}) \times A_{HN}$

S_H = nail horizontal spacing

S_V = nail vertical spacing

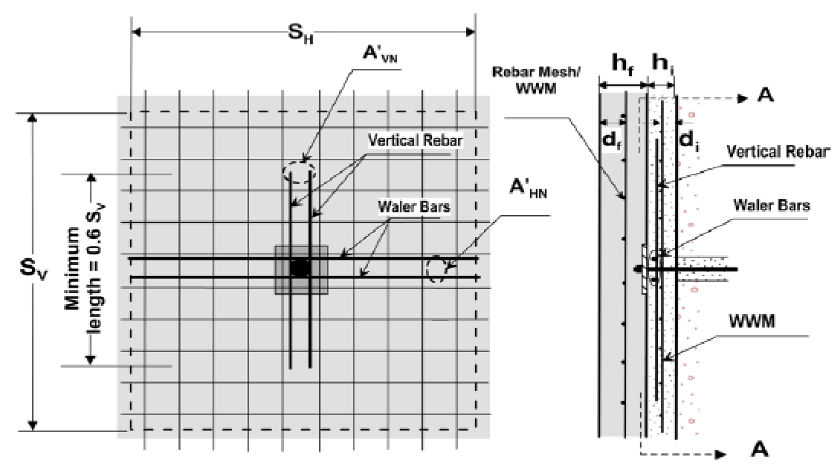
Where:

$$F = \text{smaller of} \begin{cases} (a_{vn} + a_{vm})\,[\text{in}^2/\text{ft}] \times \left(\dfrac{S_H\,h\,[\text{ft}]}{S_V}\right) \\ (a_{hn} + a_{hm})\,[\text{in}^2/\text{ft}] \times \left(\dfrac{S_V\,h\,[\text{ft}]}{S_H}\right) \end{cases}$$

Equation 6.16: Definition of F.

Table 6.5: Factor C_F

Facing	Facing Thickness, h_i or h_f (in.)	C_F
Initial	4	2.0
Initial	6	1.5
Initial	8	1.0
Final	All	1.0



Vertical Cross Sectional Area (per unit length)   Horizontal Cross Sectional Area (per unit length)

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Facing Design - Top Row of Nails - Governing Condition

Initial Facing (Shotcrete)

a_{vm} = 0.12 in^2		a_{hm} = 0.12 in^2	
$a_{vn} = a_{vm} + A_{VN}/S_H$		$a_{hn} = a_{hm} + A_{HN}/S_V$	
A_{VN} = 0.40 in^2		A_{HN} = 0.40 in^2	
S_H = 5.0 ft		S_V = 5.0 ft	
a_{vn} = 0.20 in^2/ft		a_{hn} = 0.20 in^2/ft	
$F = (a_{vn} + a_{vm}) * S_H/S_V*h$		$F = (a_{hn} + a_{hm}) * S_V/S_H*h$	
h = 0.50 ft		h = 0.50 ft	
F = smaller of: F = 0.16		**F = 0.16**	

$R_{FF} = 3.8*C_F*fy*F$

C_F = 1.50 (Table 6.5, for initial, thickness = 6.00 inches)

R_{FF} = 54.72 kips

$CDR = \phi_{FF}*R_{FF}/\gamma*To$

ϕ_{FF} = 0.90 (Table 5.9)

Seismic γ = 1.00

To = 46.97 kips

Flexural Resistance **CDR = 1.05** >= 1.00 **OK**

Check reinforcement ratios:

$$\rho_{TOT}(\%) = \left[\frac{a_{vn} + a_{vm}}{12\left(\frac{h_i}{2}\right)}\right] \times 100$$

Calculate the reinforcement ratio, ρ_{ij}, as:

$$\rho_{ij} = \frac{a_{ij}}{0.5\,h}\ 100$$

Equation 6.6: Reinforcement ratio.

Where, a_{ij} = ratio of cross-sectional area of reinforcement per unit width (in "i" direction and "j" location) and h = thickness of the facing being designed, whether initial or final. The direction "i" can be "v" (for vertical) or "h" (for horizontal); the location "j" can be "n" (nail head) or "m" (mid-span between nails).

(horizontal) ρ_{TOT} = 0.89 %

(vertical) ρ_{TOT} = 0.89 %

$$\rho_{min}[\%] = 0.24\frac{\sqrt{f'_c[psi]}}{f_y[ksi]}$$

$$\rho_{max}[\%] = 0.05\frac{f'_c[psi]}{f_y[ksi]}\left(\frac{90}{90 + f_y[ksi]}\right)$$

	ρ_{min} =	0.25 %		ρ_{max} =	2.00 %
(horizontal)	<	0.89 OK		>	0.89 OK
(vertical)	<	0.89 OK		>	0.89 OK

The additional horizontal and vertical rebar in the initial facing must be placed near the nail heads, usually within a distance h_i from the walls of the drill hole. Horizontal bars should have a length of at least $0.60S_H$; however, it is not uncommon to place waler bars with a length equal to S_H (Figure 6.5). Vertical rebar should be at least $0.60S_V$ long. Laps between adjacent bars must be in the mid-span between nails. The wire spacing in WWM typically used in soil nail walls must meet the requirements for maximum spacing of reinforcement per Section 10.6 "Distribution of Flexural Reinforcement" of ACI (2011).

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Facing Design - Top Row of Nails - Governing Condition

Final Facing (CIP Fascia)

	$a_{vm} = a_{vn}$ =	0.31	in^2	$a_{hm} = a_{hn}$ =	0.31	in^2
	S_H =	5.0	ft	S_V =	5.0	ft
	$F = (a_{vn} + a_{vm}) * S_H/S_V*h$			$F = (a_{hn} + a_{hm}) * S_V/S_H*h$		
	h =	0.75	ft	h =	0.75	ft
F = smaller of:	F =	0.47		**F =**	**0.47**	

$R_{FF} = 3.8*C_F*fy*F$
C_F = 1 (Table 6.5, for final)
R_{FF} = 106.02 kips

$CDR = \phi_{FF}*R_{FF}/\gamma*To$
ϕ_{FF} = 0.90 (Table 5.9)

Seismic:	γ =	1.00			
	To =	46.97	kips		
Flexural Resistance	**CDR =**	**1.50**	**>=**	**1.00**	**OK**

Check reinforcement ratios:

$$\rho_{TOT}(\%) = \left[\frac{a_{vn} + a_{vm}}{12\left(\frac{h_i}{2}\right)}\right] \times 100$$

(horizontal) ρ_{TOT} = 1.15 %
(vertical) ρ_{TOT} = 1.15 %

Calculate the reinforcement ratio, ρ_{ij}, as:

$$\rho_{ij} = \frac{a_{ij}}{0.5\,h}\ 100$$

Equation 6.6: Reinforcement ratio.

Where, a_{ij} = ratio of cross-sectional area of reinforcement per unit width (in "i" direction and "j" location) and h = thickness of the facing being designed, whether initial or final. The direction "i" can be "v" (for vertical) or "h" (for horizontal); the location "i" can be "n" (nail head) or "m" (mid-span between nails).

$$\rho_{min}[\%] = 0.24\,\frac{\sqrt{f'_c[psi]}}{f_y[ksi]}$$

$$\rho_{max}[\%] = 0.05\,\frac{f'_c[psi]}{f_y[ksi]}\left(\frac{90}{90 + f_y[ksi]}\right)$$

	ρ_{min} =	0.25	%	ρ_{max} =	2.00	%
(horizontal)	<	1.15	OK	>	1.15	OK
(vertical)	<	1.15	OK	>	1.15	OK

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Facing Design - Top Row of Nails - Governing Condition

Step 5 - Facing Punching Shear Resistance:

The nominal facing punching shear resistance, R_{FP}, for either situation must meet the following condition:

$$CDR = \frac{\phi_{FP} \; R_{FP}}{\gamma \; T_o} \geq 1.0$$

Equation 6.19: Capacity-to-demand ratio for punching shear resistance.

Where:

ϕ_{FP} = resistance factor for punching shear in the facing

γ = load factor selected for verification

T_o = maximum tensile force at soil nail head, as defined previously

R_{FP} is estimated as:

$$R_{FP} = C_P \; V_F$$

Equation 6.20: Nominal punching shear resistance at facing.

Where:

C_P = dimensionless factor that accounts for the contribution of the soil support under the nail head to the shear resistance

V_F = concrete punching shear basic resistance acting through the facing section

5.8.4 Punching Shear Strength Limit State

Resistance factors for punching at the facing are back-calibrated for a load factor of $\gamma_{EV} = 1.35$ for static conditions and $\gamma_{EV} = 1.00$ for seismic loading to match ASD-based designs. These values are presented in Table 5.10.

Table 5.10: Resistance Factors for Punching Shear at Facing

Condition	Case	Symbol	Resistance Factor
Static	Initial and final facing	ϕ_{FP}	0.90
Seismic loading	Initial and final facing	ϕ_{FP}	0.90

C_P can be as high as 1.15 if the soil reaction is considered. The contribution from the soil support behind the wall is conservatively assumed to be negligible; therefore, $C_P = 1.0$. Tl

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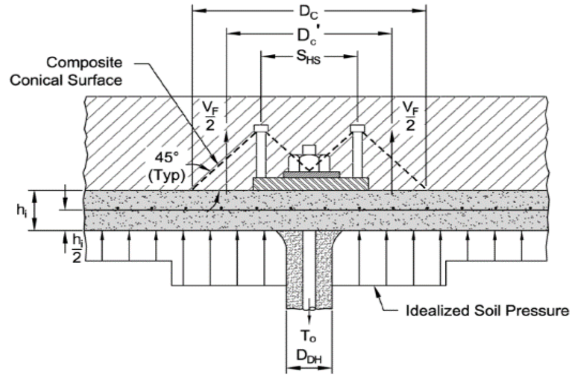
Facing Design - Top Row of Nails - Governing Condition

$$V_F\,[\text{kip}] = 0.58\,\sqrt{f'_c\ [\text{psi}]}\ \pi\ D'_c\,[\text{ft}]\,h_c\,[\text{ft}]$$

Equation 6.21: Nominal punching shear resistance through facing.



(b) Headed Stud Connection

Initial Facing (Shotcrete) — $D'c = L_{BP} + h_i$

$L_{BP} =$ 9.00 in
$hc = h_i =$ 6.00 in
$D'c =$ 15.00 in
$V_F =$ 71.99 kips

$C_P =$ 1.00 (assume negligible)
$R_{FP} = C_P{*}V_F =$ 71.99 kips

$CDR = \phi_{FP}{*}R_{FP}/\gamma{*}To$
$\phi_{FP} =$ 0.90 (Table 5.10)

Seismic: $\gamma =$ 1.00
$To =$ 46.97 kips
Punching Shear **CDR = 1.02** >= 1.00 OK

Final Facing (CIP Fascia) — $D'c =$ smaller $S_{SH}+h_c$ or $2{*}h_c$
$hc = L_S + tp - t_{SH}$

$L_S =$ 6.50 in
$tp =$ 1.0 in
$t_{SH} =$ 0.375 in
$hc =$ 7.125 in
$S_{SH} =$ 6.00 in
$S_{SH}+h_c =$ 13.125 in
$2{*}h_c =$ 14.25 in
$D'c\ (min) =$ 13.125 in
$V_F =$ 74.80 kips

$C_P =$ 1.00 (assume negligible)
$R_{FP} = C_P{*}V_F =$ 74.80 kips

$CDR = \phi_{FP}{*}R_{FP}/\gamma{*}To$
$\phi_{FP} =$ 0.90 (Table 5.9)

Seismic: $\gamma =$ 1.00
$To =$ 46.97 kips
Punching Shear **CDR = 1.06** >= 1.00 OK

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Facing Design - Top Row of Nails - Governing Condition

Step 6 - Facing Head Stud Resistance:

$$CDR = \frac{\phi_{FH}\ R_{FH}}{\gamma\ T_o} \geq 1.0$$

Equation 6.27: Capacity-to-demand ratio for a headed stud.

Where:

ϕ_{FH} = resistance factor for headed stud tensile resistance
R_{FH} = nominal tensile resistance of headed studs in final facings
γ = load factor selected for verification
T_o = as defined earlier

$$R_{FH} = N_H\ A_S\ f_{y\text{-}hs}$$

Equation 6.26: Headed stud resistance at facing.

Where:

N_H = number of headed studs in the connection (usually 4)
A_S = cross-sectional area of the headed stud shaft
$f_{y\text{-}hs}$ = tensile yield strength of headed stud

5.8.5 Headed Stud in Tension in Final Facing

Resistance factors for headed stud in tension punching in the final facing are back-calibrated for a load factor of γ_{EV} = 1.35 for static conditions and γ_{EV} = 1.00 for seismic loading to match ASD-based designs. These values are presented in Table 5.11.

Table 5.11: Resistance Factors for Headed Stud in Tension in Final Facings

Condition	Case	Symbol	Resistance Factor
Static	A307 Steel Bolt (1)	ϕ_{FH}	0.70
Static	A325 Steel Bolt	ϕ_{FH}	0.80
Seismic loading	A307 Steel Bolt (1)	ϕ_{FH}	0.65
Seismic loading	A325 Steel Bolt	ϕ_{FH}	0.75

Note: (1) This is equivalent to AWS D1.1 Type B studs, with f_y = 60 ksi.

N_H = 4
As = 0.44 in^2
fy = 60.00 ksi
R_{FH} = 105.98 kips

$CDR = \phi_{FP} * R_{FP} / \gamma * To$

ϕ_{FP} = 0.70 (Table 5.11)

Seismic: γ = 1.00
To = 46.97 kips

Stud Resistance **CDR = 1.17** >= 1.00 OK

Summary

Shotcrete Layer:
thickness = 6.00 in
main reinf = 4x4 - W4.0 x W4.0 — Horz bar L = 0.6Sh = 3.00 ft
bar size at nails = 4 (2 vert and 2 horz) — Vert bar L = 0.6Sv = 3.00 ft

CIP Fascia
thickness = 9.00 in
bar size 5 spaced at 12.0 in

Top Row Bearing Plate
square - size = 9.0 x 9.0 x 1.0 in thick
4 anchors spaced at = 6.00 in
anchor L = 6.50 in
anchor D = 0.75 in

GOVERNS FOR FINAL THICKNESS OF SHOTCRETE AND CIP CONCRETE

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Concrete Stem Analysis

Input

stem ht =	9.00	ft	(max at Wall Station 2+38 step)
Coping above stem =	0.83	ft	
Rail ht =	3.50	ft	
Depth to first nail =	2.00	ft	(typ 2.0 ft from top)
Analysis H =	15.33	ft	
stem width =	0.75	ft	(ignoring extra thickness on top of shotcrete)

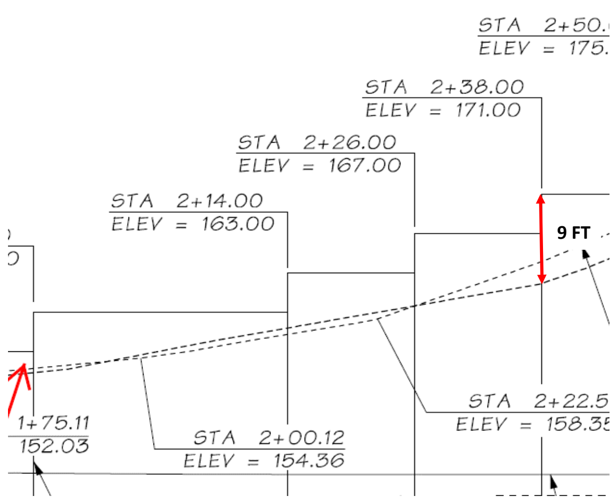
Load Factors

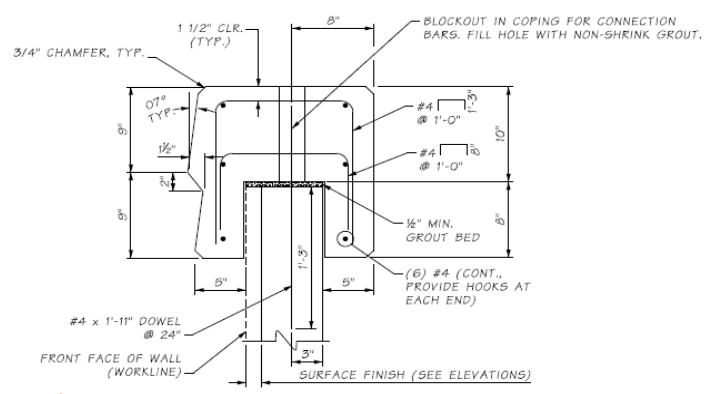
	AASHTO LRFD T 3.4.1-1		
Design check with Strength I limit state:	LL	1.75	(WAC fall protection rail load only)
Design check with Strength V limit state:	LL	1.35	(WAC fall protection rail and wind load)
Design check with Strength III limit state:	WS	1.00	(Wind only)
Design check with Extreme Event I limit state:	EQ	1.00	(Seismic only)

WAC Fall Protection Load

	F = 0.20	k (unfactored)	
Strength I	0.35	k (factored)	(for check with no other loads)
Strength V	0.27	k (factored)	(for check with wind)

Loading would transfer into the wall via the wall connection/rebar spaced at an interval to resist the loading between the max 8-ft post locations. Assume 2 closest bars (connection bars to be spaced at 24") to post location take half loading each (remaining portion of wall does not help to resist loads). See wall cap connection calculations for connection bar design.





F =	0.10	k (unfactored)	(F/2)
	0.175	k (factored)	(Strength I)
	0.135	k (factored)	(Strength V)

JOB NO.	PS19203160	SHEET 45 OF 51		
PHASE	Design	TASK	Wall 09.05R-B	
JOB NAME	I-405; Renton To Belleview Widening and Express Toll Lanes Project			Colorado Center Tower II
BY	JDF	DATE	8/7/2020	2000 S. Colorado Blvd., Ste 2-1000
CHECKED BY	DMP	DATE	9/24/2020	Denver, CO 80222
				+1 (303) 935-6505 Fax '+1 (303) 935-6575

Materials Input

f'c = 4.0 ksi — BDM 5.1.1.B.2
β1 = 0.85 — And per Plan Sheets

fy = 60.0 ksi — BDM 5.1.2

γ_e = 1 — AASHTO LRFD 5.6.7

modulus of rupture coefficient = 0.24 — AASHTO LRFD 5.4.2.6
fr = 0.480 ksi $\quad 0.24\,\lambda\,\sqrt{f'_c}$

wc = 0.155 kcf — BDM 3.8

Ec = 4576 ksi $\quad E_c = 120{,}000 K_1 w_c^{2.0} f'^{0.33}_c$ — AASHTO LRFD 5.4.2.4-1

n = $\frac{29000}{4576}$ = 6.34

λ = 1.0 — AASHTO LRFD 5.4.2.8

γ1 = 1.60 — AASHTO LRFD 5.6.3.3
γ3 = 0.75 (per notes for ASTM A706, Grade 60)

Structural Design Loads

Analysis H = 15.33 ft (height above fixed point of stem where load acts)

Strength I (WAC Fall Protection)

Vu = 0.18 k

Mu = 2.68 k-ft (Analysis Ht * Vu)

Minimum Reinforcement

Minimum Reinforcement — AASHTO LRFD 5.6.3.3
1.33 Mu = 3.57 k-ft

S = I/c = $\frac{=1/12\,(b)\,(h)^3 =}{c}$ $\frac{729}{4.5}$ = 162 in^3

$$Mcr = \frac{\overset{\gamma 1}{1.60} \times \overset{\gamma 3}{0.75} \times \overset{fr}{0.48} \times \overset{S}{162}}{12} = 7.78 \text{ k-ft}$$

Strength I Loading Summary

Vu = 0.18 k
Mu = 3.57 k-ft =MAX(Mu, MIN(1.33Mu, Mcr))

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Strength V (WAC Fall Protection and Wind)

WAC Fall Protection

Vu = 0.14 k

Mu = 2.07 k-ft (Analysis Ht * Vu)

Wind Loading

P_Z = 0.0197 ksf (see Intro sheets for development)

apply as liner pressure over H = 13.33 ft (rail h = 3.5' + coping 0.83' + panel h = 9')

Vu = 0.26 kips (factor = 1)

Acting at 0.55*H + 2 ft to Nail Connection = 9.33 ft

Ms/Mu = 2.45 k-ft

Combined

Vu = 0.40 k

Mu = 4.52 k-ft

Minimum Reinforcement

Minimum Reinforcement AASHTO LRFD 5.6.3.3

1.33 Mu = 6.01 k-ft

$$S = I/c = \frac{=1/12\ (b)\ (h)^\wedge 3 =}{c} \quad \frac{729}{4.5} = 162 \text{ in}^3$$

$$Mcr = \frac{\overset{\gamma 1}{1.60} \times \overset{\gamma 3}{0.75} \times \overset{fr}{0.48} \times \overset{S}{162}}{12} = 7.78 \text{ k-ft}$$

Strength V Loading Summary

Vu = 0.40 k

Mu = 6.01 k-ft =MAX(Mu, MIN(1.33Mu, Mcr))

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Strength III (Wind Loading)

Wind Loading

P_Z = 0.0316 ksf (see Intro sheets for development)
apply as liner pressure over H = 13.33 ft (rail h = 3.5' + coping 0.83' + panel h = 9')
Vu = 0.42 kips (factor = 1)

Acting at 0.55*H + 2 ft to Nail Connection = 9.33 ft
Mu = 3.93 k-ft

Minimum Reinforcement

Minimum Reinforcement AASHTO LRFD 5.6.3.3

1.33 Mu = 5.23 k-ft

$$S = I/c = \frac{=1/12\ (b)\ (h)^3 =}{c} \quad \frac{729}{4.5} = 162 \ \text{in}^3$$

$$Mcr = \frac{\overset{\gamma1}{1.60} \times \overset{\gamma3}{0.75} \times \overset{fr}{0.48} \times \overset{S}{162}}{12} = 7.78 \ \text{k-ft}$$

Strength III Loading Summary

Vu = 0.42 k
Mu = 5.23 k-ft =MAX(Mu, MIN(1.33Mu, Mcr))

Extreme Event I (Seismic)

Provided - see separate seismic calculations
Vu = 3.93 k
Mu = 6.24 k-ft

Service I (Wind 70mph and Normal Operating Loads)

Wind Loading

P_Z = 0.015 ksf (see Intro sheets for development)
apply as liner pressure over H = 13.33 ft (rail h = 3.5' + coping 0.83' + panel h = 9')
Vs = 0.20 kips (factor = 1)

Acting at 0.55*H + 2 ft to Nail Connection = 9.33 ft
Ms = 1.87 k-ft

WAC Loading (Service I, LL factor = 1)

Vs = 0.10 k
ht = 15.33 ft
Ms = 1.53 k-ft (Analysis Ht * Vu)

Service I Loading Summary

Vs = 0.30 k
Ms = 3.40 k-ft

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wood.

Flexural Design

Strength: Φ Flexure = 0.9 assumes tension controlled — AASHTO LRFD 5.5.4.2
Extreme: 1.0 for seismic loads

Stem

h = 9.00 in
center of bars = 4.5 in
Bar size = 5
0.31 sq in / bar

$$de = 9.00 - 4.50 + \frac{5}{16} = 4.81 \text{ in}$$

$$dc = 4.50 + \frac{5}{16} = 4.81 \text{ in}$$

Bar spacing = 12 in
As / ft = 0.31 sq in / ft

$$\text{stress block depth, } a = \frac{As \times fy}{0.85 \times f'c \times 12} = 0.46 \text{ in}$$

Strength: Φ Mn = 0.90 x As x fy (de - a/2) = 6.40 k-ft > 6.01 OK
Extreme: Φ Mn = 1.00 x As x fy (de - a/2) = 7.11 k-ft > 6.24 OK

Crack Control

AASHTO LRFD 5.6.7

Crack control is required where the tension in the section exceeds 80% modulus of rupture (fr) = 0.480 ksi
80%fr = 0.384 ksi
Compute section area moment to equal criteria above and compare to moment from loading:
M*c/I = 0.384 S = I/c = 162 in^3
M = 5.184 k-ft > 3.40 OK **Do not need to provide crack control**

Stem

reinforcement ratio $\rho = \frac{As}{b\ de} = \frac{0.31}{12.00 \times 4.81} = 0.0054$

$$k = \sqrt{2n\rho + (n\rho)^2} - n\rho = 0.229$$

$$j = 1 - k/3 = 0.92$$

$$fss = \text{Min}\left(0.6\,Fy, \frac{Mserv}{As\,(j)\,de}\right) = 29.60 \text{ ksi}$$

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} = 2.64$$ AASHTO LRFD 5.6.7-2

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c = -0.67 \text{ in}$$ AASHTO LRFD 5.6.7-1

< 12 NG<-

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Shear Design

AASHTO LRFD 5.7.3.3

Strength: Φ Shear = 0.90
Extreme: 1.0 for seismic loads

β = 2.00
θ = 45° — AASHTO LRFD 5.5.4.2
bv = 12.00 in — AASHTO LRFD 5.7.3.4.1

Stem

0.9 de = 0.90 x 4.81 = 4.33 in
0.72 h = 0.72 x 9.00 = 6.48 in — AASHTO LRFD 5.7.2.8
dv = max(0.9 de, 0.72 h) = 6.48 in

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}\, b_v d_v$$

AASHTO LRFD 5.7.3.3-3

Vc = 9.83 k
Strength: Φ Vn = 8.85 k > 0.42 OK
Extreme: Φ Vn = 9.83 k > 3.93 OK

Summary

	Bar #		Spa		
Bars:	5	@	12	in	(in panels above top row of nails)

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Additional Checks

Temp Shrinkage in Wall Panels above grade

- reinforcement is provided for front face of CIP, check meets min requirements, and apply As for reinforcement for back face of wall for temp/shrinkage.

Check max panel:

stem h = 9.00 ft (max at Wall Station 2+38 step)
108.00 in
stem b = 1.25 ft
15.00 in
fy = 60.00 ksi BDM 5.1.2

As ≥ 0.14 in^2 each way on back face

Provide Bar = 4
As = 0.20 sq in / bar > 0.14 OK

Spacing:
thickness less than 18"
spa ≤ 18 in
spa ≤ 3*b
45 in
max spa = 18 in

5.10.6—Shrinkage and Temperature Reinforcement

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete. Temperature and shrinkage reinforcement to ensure that the total reinforcement on exposed surfaces is not less than that specified herein.

Reinforcement for shrinkage and temperature may be in the form of bars, welded wire reinforcement, or prestressing tendons.

For bars or welded wire reinforcement, the area of reinforcement per foot, on each face and in each direction, shall satisfy the following:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} \qquad (5.10.6\text{-}1)$$

except that:

$$0.11 \leq A_s \leq 0.60 \qquad (5.10.6\text{-}2)$$

where:

A_s = area of reinforcement in each direction and each face (in.²/ft)
b = least width of component section (in.)
h = least thickness of component section (in.)
f_y = specified minimum yield strength of reinforcement ≤75.0 ksi

Where the least dimension varies along the length of wall, footing, or other component, multiple sections should be examined to represent the average condition at each section. Spacing shall not exceed the following:

- 12.0 in. for walls and footings greater than 18.0 in. thick
- 12.0 in. for other components greater than 36.0 in. thick
- For all other situations, 3.0 times the component thickness but not less than 18.0 in.

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Development Length WWF in shotcrete for lap lengths

ASTM STANDARD WIRE REINFORCEMENT

W & D size		Nominal diameter, in.	Nominal area, in.^{2}	Nominal weight, lb/ft	Area, in.^{2}/ft of width for various spacings Center-to-center spacing, in.						
Plain	Deformed				2	3	4	6	8	10	12
W4	D4	0.226	0.040	0.136	0.24	0.16	0.12	0.08	0.06	0.04	0.04

$$\ell_{hd} \geq 0.95 d_b \frac{f_y - 20.0}{\lambda\sqrt{f'_c}}, \qquad (5.10.8.2.5a\text{-}1)$$

db =	0.23	in	W4
fy =	60.00	ksi	BDM 5.1.2
λ =	1.00		AASHTO 5.4.2.8)
f'c =	4.00	ksi	BDM 5.1.1.B.2
√f'c =	2	ksi	
l_{hd} =	4.294	in	

$$\ell_{hd} \geq 6.30 \frac{A_w f_y}{s_w \lambda\sqrt{f'_c}} \qquad (5.10.8.2.5a\text{-}2)$$

Aw =	0.04		W4
fy =	60.00	ksi	BDM 5.1.2
sw =	4.00	in	
λ =	1.00		
f'c =	4.00	ksi	
√f'c =	2.00	ksi	
l_{hd} =	1.89	in	

l_{hd} = 4.29 in (max)

5.10.8.5—Splices of Welded Wire Reinforcement

5.10.8.5.1—Splices of Welded Deformed Wire Reinforcement in Tension

When measured between the ends of each fabric sheet, the length of lap for lap splices of welded deformed wire reinforcement with cross wires within the lap length shall not be less than the greater of the following:

- $1.3\ell_{hd}$; or
- 8.0 in.

The overlap measured between the outermost cross wires of each reinforcement sheet shall not be less than 2.0 in.

Lap splices of welded deformed wire reinforcement with no cross wires within the lap splice length shall be determined as for deformed wire in accordance with the provisions of Article 5.10.8.4.3a.

min lap = 1.3*l_{hd}

min lap =	6	in
Or =	8	in (min)

lap = 8 in

5.10.8.2.5—Welded Wire Reinforcement

5.10.8.2.5a—Welded Deformed Wire Reinforcement

For applications other than shear reinforcement, the basic development length, ℓ_{hd}, in in., of welded deformed wire reinforcement, measured from the point of critical section to the end of wire, shall not be less than the greater of the following:

- the product of the basic development length and the applicable modification factor or factors, as specified in Article 5.10.8.2.2b; and
- 8.0 in., except for lap splices, as specified in Article 5.10.8.5.1.

The development of shear reinforcement shall be taken as specified in Article 5.10.8.2.6.

The basic development length, ℓ_{hd}, for welded deformed wire reinforcement, with not less than one cross wire within the development length at least 2.0 in. from the point of critical section, shall satisfy the following:

$$\ell_{hd} \geq 0.95 d_b \frac{f_y - 20.0}{\lambda\sqrt{f'_c}}, \qquad (5.10.8.2.5a\text{-}1)$$

$$\ell_{hd} \geq 6.30 \frac{A_w f_y}{s_w \lambda\sqrt{f'_c}} \qquad (5.10.8.2.5a\text{-}2)$$

where:

A_w = area of an individual wire to be developed or spliced (in.^{2})
s_w = spacing of wires to be developed or spliced (in.)
d_b = nominal diameter of reinforcing bar or wire (in.)
f'_c = compressive strength of concrete for use in design not to be taken greater than 15.0 ksi for normal weight concrete and 10.0 ksi for lightweight concrete (ksi)
f_y = specified minimum yield strength of reinforcement (ksi)
λ = concrete density modification factor as specified in Article 5.4.2.8

The basic development length of welded deformed wire reinforcement, with no cross wires within the development length, shall be determined as for deformed wire in accordance with Article 5.10.8.2.1a.

5.4.2.8—Concrete Density Modification Factor

The concrete density modification factor, λ, shall be determined as:

- Where the splitting tensile strength, f_{ct}, is specified:

$$\lambda = 4.7 \frac{f_{ct}}{\sqrt{f'_c}} \leq 1.0 \qquad (5.4.2.8\text{-}1)$$

- Where f_{ct} is not specified:

$$0.75 \leq \lambda = 7.5\, w_c \leq 1.0 \qquad (5.4.2.8\text{-}2)$$

- Where normal weight concrete is used, λ shall be taken as 1.0.

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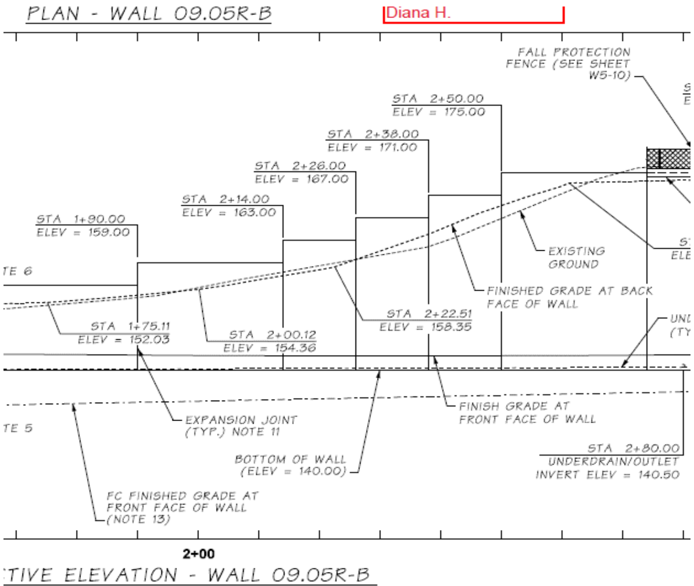
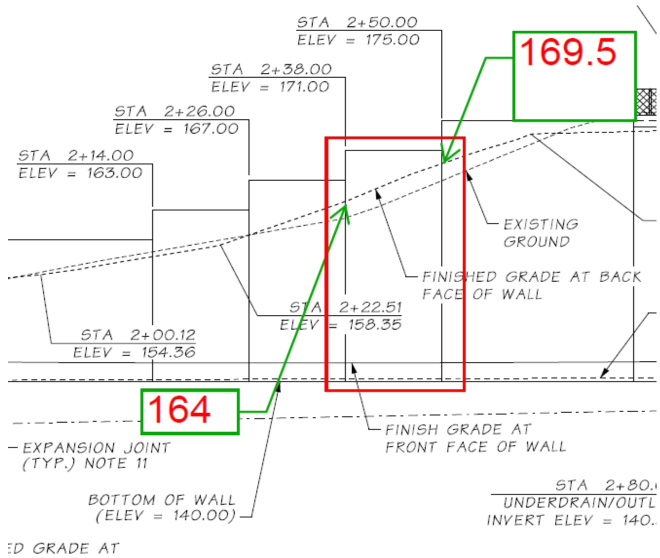
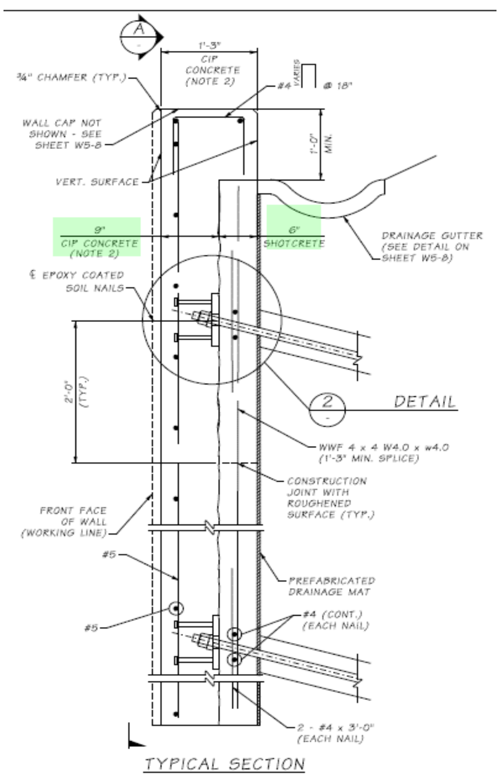
PHASE	Final Design	TASK	Wall 09.05R-B	
JOB NAME	I-405 Design Build			Colorado Center Tower II
BY	DMP	DATE	9/21/2020	2000 S. Colorado Blvd., Ste 2-1000
CHECKED BY	EHP	DATE	9/22/2020	Denver, CO 80222
				+1 (303) 935-6505 Fax '+1 (303) 935-6575

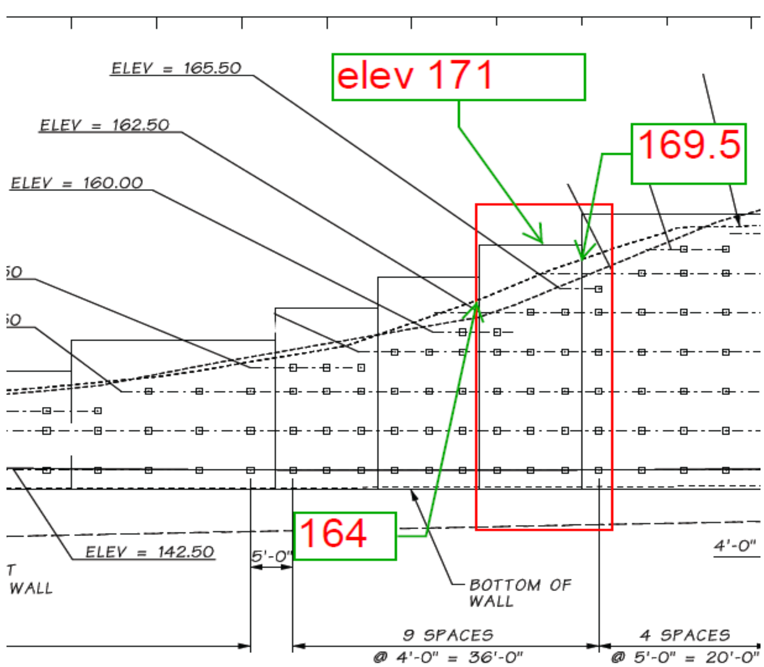
Introduction

Determine the seismic loading on the above grade portion of soil nail wall 09.05R-B using LARSA
Model the wall using plate elements. Fix the wall at the base.









JOB NO.		SHEET	2 OF 9
PHASE	Final Design	TASK	Wall 09.05R-B
JOB NAME	I-405 Design Build		
BY	DMP	DATE	9/21/2020
CHECKED BY	EHP	DATE	9/22/2020

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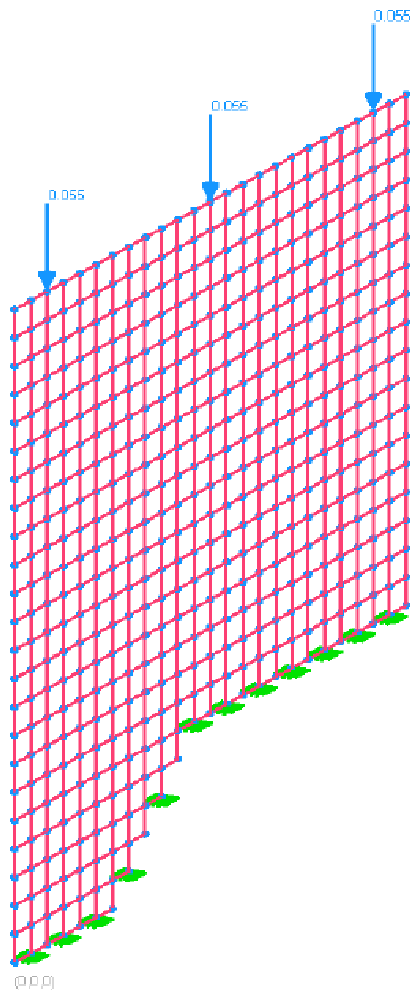
Model Overall View



LARSA Input

Material Properties

Properties: Material Properties

Materials | Sections | UCS | Spring Properties | Isolator Definitions | Temperature Curve

Basic Properties | More Properties

	Name	Modulus of Elasticity (kips/in²)	Poisson Ratio	Shear Modulus (kips/in²)	Unit Weight (kips/in³)	Thermal Expansion (1/ °F	Assigned
1	Fc_4	3,605.00	0.1697	1,541.00	0.0001	5.500000	Yes
2							

JOB NO.		SHEET	3 OF 9
PHASE	Final Design	TASK	Wall 09.05R-B
JOB NAME	I-405 Design Build		
BY	DMP	DATE	9/21/2020
CHECKED BY	EHP	DATE	9/22/2020

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Geometry

Plates

Joints | Members | Plates | Springs | Mass Elements | Isolators | Tendons | Lanes/Surfaces

Plates | Plate Offsets

	ID	Bending Type	Membrane Type	I-Joint	J-Joint	K-Joint	L-Joint	Material	Thickness (in)	Area (ft²)	Casting (day)	Structure Grou
1	1	Thin Plate	Drilling / CST	1	2	25	24	Fc_4	11.9100	0.2500	0	(none)
2	2	Thin Plate	Drilling / CST	2	3	26	25	Fc_4	11.9100	0.2500	0	(none)
3	3	Thin Plate	Drilling / CST	3	4	27	26	Fc_4	11.9100	0.2500	0	(none)
4	4	Thin Plate	Drilling / CST	4	5	28	27	Fc_4	11.9100	0.2500	0	(none)
5	5	Thin Plate	Drilling / CST	5	6	29	28	Fc_4	11.9100	0.2500	0	(none)
461	482	Thin Plate	Drilling / CST	510	511	530	529	Fc_4	11.9100	0.2500	0	(none)
462	483	Thin Plate	Drilling / CST	511	512	531	530	Fc_4	11.9100	0.2500	0	(none)
463	484	Thin Plate	Drilling / CST	512	513	532	531	Fc_4	11.9100	0.2500	0	(none)
464	485	Thin Plate	Drilling / CST	513	514	533	532	Fc_4	11.9100	0.2500	0	(none)
465	486	Thin Plate	Drilling / CST	514	515	534	533	Fc_4	11.9100	0.2500	0	(none)
466	487	Thin Plate	Drilling / CST	515	516	535	534	Fc_4	11.9100	0.2500	0	(none)
467	488	Thin Plate	Drilling / CST	516	517	536	535	Fc_4	11.9100	0.2500	0	(none)
468	489	Thin Plate	Drilling / CST	517	518	537	536	Fc_4	11.9100	0.2500	0	(none)
469	490	Thin Plate	Drilling / CST	518	519	538	537	Fc_4	11.9100	0.2500	0	(none)
470	491	Thin Plate	Drilling / CST	519	520	539	538	Fc_4	11.9100	0.2500	0	(none)
471	492	Thin Plate	Drilling / CST	520	521	540	539	Fc_4	11.9100	0.2500	0	(none)
472	493	Thin Plate	Drilling / CST	121	122	146	541	Fc_4	11.9100	0.2500	0	(none)

Plate area = 0.5 x 0.5 = 0.25 sq ft

Thickness = 11.91 in, okay, adjusted to account for cracked section

Wall length = 250.00 - 238.00 = 12.00 ft 12' in larsa, OK

1 Wall ht = 171.00 - 160.00 = 11.00 ft 11.5' in larsa, OK
2 Wall ht = 171.00 - 162.50 = 8.50 ft 9.0' in larsa, OK

1 Length = 3.00 ft Ok
2 Length = 2.00 ft Ok
3 Length = 7.00 ft Ok

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PHASE	Final Design	TASK	Wall 09.05R-B
JOB NAME	I-405 Design Build		
BY	DMP	DATE	9/21/2020
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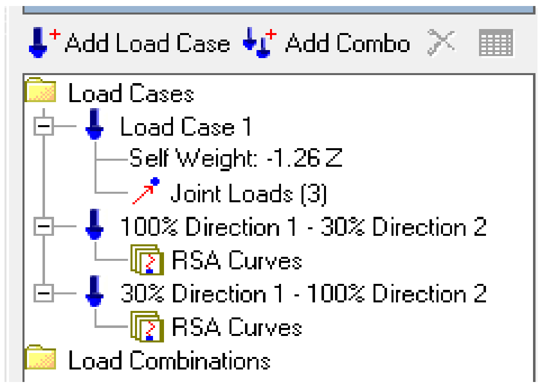
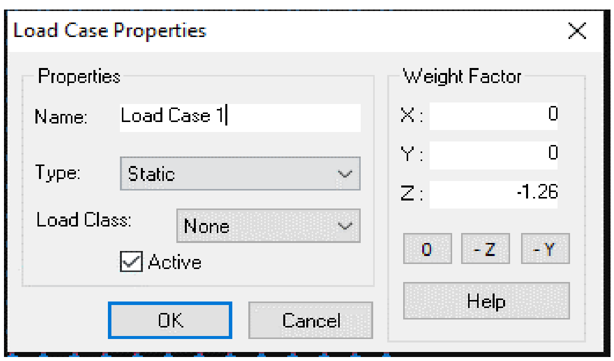
Loads



Self Wt.



wt factor, z = -1.26 accounts for plate thickness adjustment

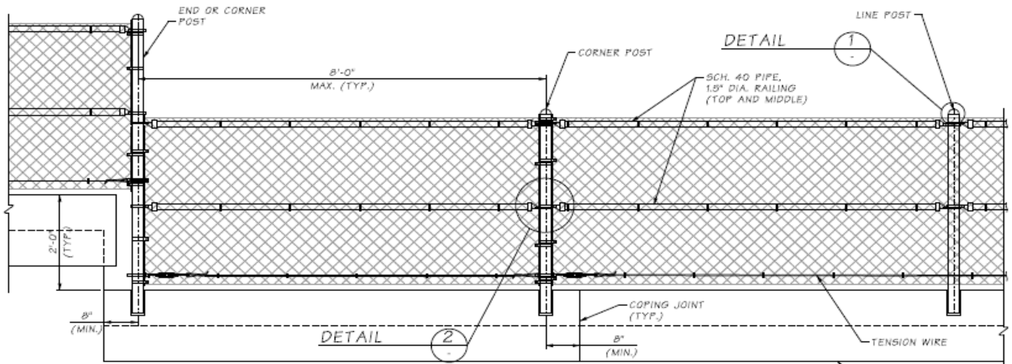
JOB NO.		SHEET	5 OF 9	wood.
PHASE	Final Design	TASK	Wall 09.05R-B	
JOB NAME		I-405 Design Build		Colorado Center Tower II
BY	DMP	DATE	9/21/2020	2000 S. Colorado Blvd., Ste 2-1000
CHECKED BY	EHP	DATE	9/22/2020	Denver, CO 80222
				+1 (303) 935-6505 Fax '+1 (303) 935-6575

Fence Load

Load Case 1:Joint Loads

Joint Loads | Support Disp | Member Loads | Plate Loads | Moving Loads | Time History | RSA Loads

	Joint	X-Force (kips)	Y-Force (kips)	Z-Force (kips)	X-Moment (kips-ft)	Y-Moment (kips-ft)	Z-Moment (kips-ft)
1	72	0.0000	0.0000	-0.0550	0.0000	0.0000	0.0000
2	312	0.0000	0.0000	-0.0550	0.0000	0.0000	0.0000
3	502	0.0000	0.0000	-0.0550	0.0000	0.0000	0.0000
4							



TOP MOUNT FALL PROTECTION FENCE DETAIL - 2-FT STEP

Total fence wt =	0.055	x	3	=	0.165	k
wt / ft =	0.165	/	12.00	=	0.014	k / ft, reasonable

JOB NO.		SHEET	6 OF 9
PHASE	Final Design	TASK	Wall 09.05R-B
JOB NAME	I-405 Design Build		
BY	DMP	DATE	9/21/2020
CHECKED BY	EHP	DATE	9/22/2020

wood.
Colorado Center Tower II
2000 S. Colorado Blvd., Ste 2-1000
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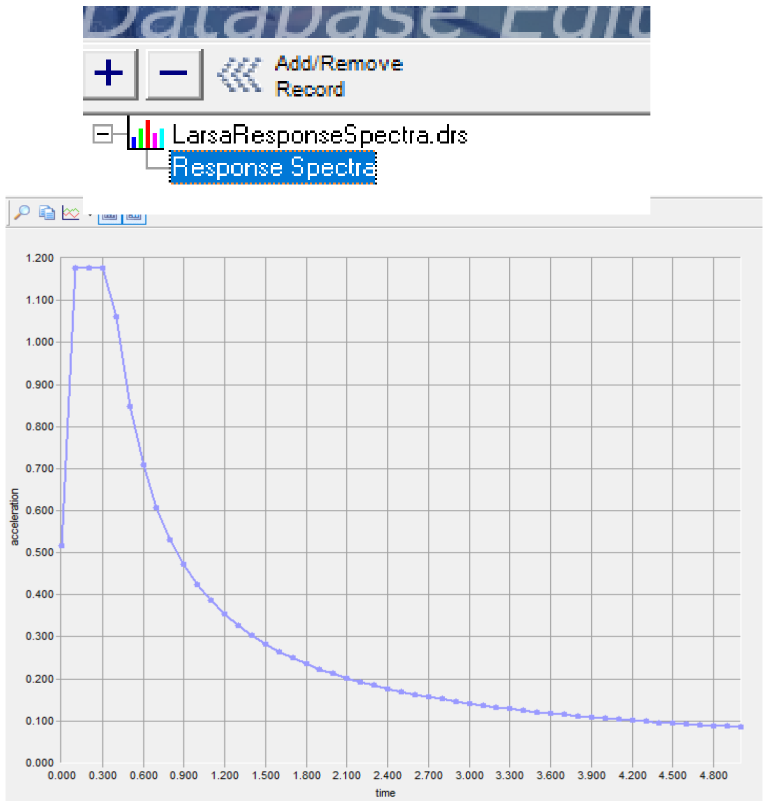
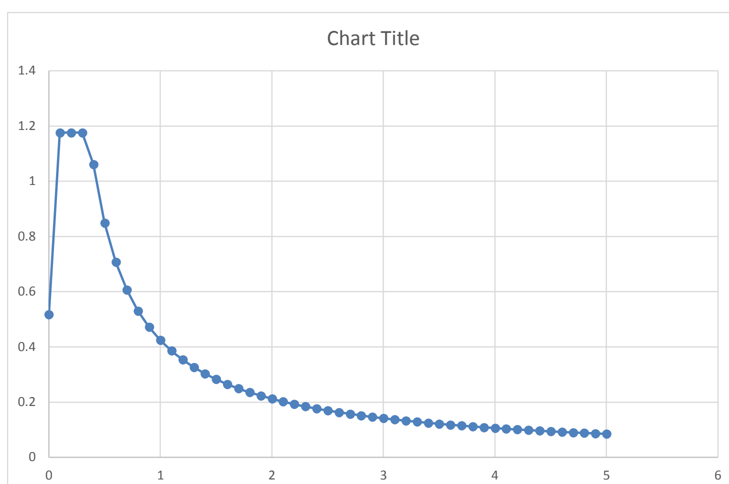
RSA Curves / Loads

Response Spectra

Time (sec)	Acceleration (ft/s²)		
0	0.517	0.5172	1.0
0.1	1.176	1.1760	1.0
0.2	1.176	1.1760	1.0
0.3	1.176	1.1760	1.0
0.4	1.061	1.0613	1.0
0.5	0.849	0.8490	1.0
0.6	0.708	0.7075	1.0
0.7	0.606	0.6064	1.0
0.8	0.531	0.5306	1.0
0.9	0.472	0.4717	1.0
1	0.425	0.4245	1.0
1.1	0.386	0.3859	1.0
1.2	0.354	0.3538	1.0
1.3	0.327	0.3265	1.0
1.4	0.303	0.3032	1.0
1.5	0.283	0.2830	1.0
1.6	0.265	0.2653	1.0
1.7	0.25	0.2497	1.0
1.8	0.236	0.2358	1.0
1.9	0.223	0.2234	1.0
2	0.212	0.2123	1.0
2.1	0.202	0.2021	1.0
2.2	0.193	0.1930	1.0
2.3	0.185	0.1846	1.0
2.4	0.177	0.1769	1.0
2.5	0.17	0.1698	1.0
2.6	0.163	0.1633	1.0
2.7	0.157	0.1572	1.0
2.8	0.152	0.1516	1.0
2.9	0.146	0.1464	1.0
3	0.142	0.1415	1.0
3.1	0.137	0.1369	1.0
3.2	0.133	0.1327	1.0
3.3	0.129	0.1286	1.0
3.4	0.125	0.1249	1.0
3.5	0.121	0.1213	1.0
3.6	0.118	0.1179	1.0
3.7	0.115	0.1147	1.0
3.8	0.112	0.1117	1.0
3.9	0.109	0.1088	1.0
4	0.106	0.1061	1.0
4.1	0.104	0.1035	1.0
4.2	0.101	0.1011	1.0
4.3	0.099	0.0987	1.0
4.4	0.096	0.0965	1.0
4.5	0.094	0.0943	1.0
4.6	0.092	0.0923	1.0
4.7	0.09	0.0903	1.0
4.8	0.088	0.0884	1.0
4.9	0.087	0.0866	1.0
5	0.085	0.0849	1.0





JOB NO.		SHEET	7 OF 9
PHASE	Final Design	TASK	Wall 09.05R-B
JOB NAME	I-405 Design Build		
BY	DMP	DATE	9/21/2020
CHECKED BY	EHP	DATE	9/22/2020

wood.
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3.10.4—Seismic Hazard Characterization

3.10.4.1—Design Response Spectrum

The five-percent-damped-design response spectrum shall be taken as specified in Figure 3.10.4.1-1. This spectrum shall be calculated using the mapped peak ground acceleration coefficients and the spectral acceleration coefficients from Figures 3.10.2.1-1 to 3.10.2.1-21, scaled by the zero-, short-, and long-period site factors, F_{pga}, F_a, and F_v, respectively.

3.10.8—Combination of Seismic Force Effects

The elastic seismic force effects on each of the principal axes of a component resulting from analyses in the two perpendicular directions shall be combined to form two load cases as follows:

AASHTO 3.10

- 100 percent of the absolute value of the force effects in one of the perpendicular directions combined with 30 percent of the absolute value of the force effects in the second perpendicular direction, and
- 100 percent of the absolute value of the force effects in the second perpendicular direction combined with 30 percent of the absolute value of the force effects in the first perpendicular direction.

100% Direction 1 - 30% Direction 2

100% Direction 1 - 30% Direction 2:RSA Parameters

Joint Loads | Support Disp | Member Loads | Plate Loads | Moving Loads | Time History | **RSA Loads**

	Name	Value
1	Response-Spectrum Curve in Direction 1	Response Spectra
2	Scale in Direction 1	1.0000
3	Response-Spectrum Curve in Direction 2	Response Spectra
4	Scale in Direction 2	0.3000
5	Response-Spectrum Curve in Global Z	(none)
6	Scale in Global Z	0.0000
7	Angle from Global X to Direction 1	0.0000
8	Modal Combination Method	CQC
9	Spatial Combination Method	SRSS
10	Modal Damping Ratio (if no curve)	0.0500
11	Sign Assigned to Results	No Sign
12	Modal Combination System	Global
13	Modal Damping Curve	(none)
14		

30% Direction 1 - 100% Direction 2

30% Direction 1 - 100% Direction 2:RSA Parameters

Joint Loads | Support Disp | Member Loads | Plate Loads | Moving Loads | Time History | **RSA Loads**

	Name	Value
1	Response-Spectrum Curve in Direction 1	Response Spectra
2	Scale in Direction 1	0.3000
3	Response-Spectrum Curve in Direction 2	Response Spectra
4	Scale in Direction 2	1.0000
5	Response-Spectrum Curve in Global Z	(none)
6	Scale in Global Z	0.0000
7	Angle from Global X to Direction 1	0.0000
8	Modal Combination Method	CQC
9	Spatial Combination Method	SRSS
10	Modal Damping Ratio (if no curve)	0.0500
11	Sign Assigned to Results	No Sign
12	Modal Combination System	Global
13	Modal Damping Curve	(none)
14		

JOB NO.		SHEET	8 OF 9	wood.
PHASE	Final Design	TASK	Wall 09.05R-B	
JOB NAME	I-405 Design Build			Colorado Center Tower II
BY	DMP	DATE	9/21/2020	2000 S. Colorado Blvd., Ste 2-1000
CHECKED BY	EHP	DATE	9/22/2020	Denver, CO 80222
				+1 (303) 935-6505 Fax '+1 (303) 935-6575

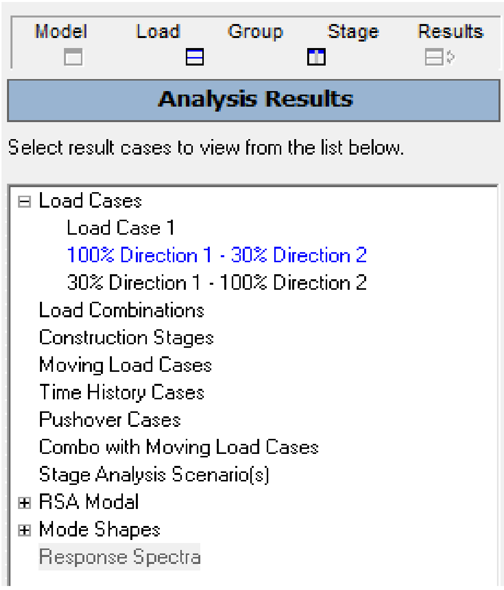
Number of Modes

	Mode Shape	Cumulative Per	Cumulative Per	Cumulative Per Mass Z	
1	M1: f = 16.9, t =	1.17E-23	62.64035	3.00E-23	
2	M2: f = 35.58, t	1.22E-23	62.73439	3.17E-23	
3	M3: f = 89.3, t =	1.58E-21	72.16426	1.57E-21	
4	M4: f = 105.94,	2.78E-21	80.56821	1.69E-21	
5	M5: f = 112.03,	63.71391	80.56821	1.212071	
6	M6: f = 137.4, t	63.71391	82.61083	1.212071	
7	M7: f = 213.42,	63.71391	83.45563	1.212071	
8	M8: f = 224.45,	63.71391	83.67841	1.212071	
9	M9: f = 244.97,	65.72909	83.67841	84.81549	
10	M10: f = 256.49	65.72909	87.0851	84.81549	
11	M11: f = 294.92	86.38684	87.0851	86.19146	
12	M12: f = 339.5,	86.38684	88.83591	86.19146	
13	M13: f = 357.19	86.38684	89.25755	86.19146	
14	M14: f = 383.15	86.38934	89.25755	86.23537	
15	M15: f = 411.81	86.38934	89.3239	86.23537	
16	M16: f = 414.78	86.38934	89.60686	86.23537	
17	M17: f = 451.33	87.19411	89.60686	87.1981	
18	M18: f = 465.9,	87.19411	91.25784	87.1981	
19	M19: f = 480.94	89.91949	91.25784	87.32425	
20	M20: f = 523.83	89.91949	91.25819	87.32425	
21	M21: f = 577.34	89.91949	91.3662	87.32425	
22	M22: f = 595.25	92.6595	91.3662	87.38086	
23	M23: f = 653.69	92.6595	92.10832	87.38086	
24	M24: f = 655.21	92.97517	92.10832	90.40788	
25	M25: f = 670.7,	92.97517	92.12008	90.40788	
26	M26: f = 679.93	93.54955	92.12008	90.45209	
27	M27: f = 688.0,	93.54955	92.58095	90.45209	
28	M28: f = 701.69	93.5629	92.58095	91.60355	
29	M29: f = 721.64	93.5629	93.33189	91.60355	
30	M30: f = 780.56	95.05161	93.33189	92.97468	
31	M31: f = 781.31	95.05161	93.58736	92.97468	
32	M32: f = 796.48	95.20058	93.58736	95.48043	
33	M33: f = 811.81	95.20058	93.62005	95.48043	
34	M34: f = 840.96	95.20058	93.70685	95.48043	
35	M35: f = 856.96	95.35259	93.70685	95.53812	35 modes OK for cumulative mass
36	M36: f = 875.72	95.3652	93.70685	95.73635	
37	M37: f = 916.37	95.7075	93.70685	95.75085	
38	M38: f = 946.76	95.85549	93.70685	95.75243	
39	M39: f = 957.36	95.85549	93.71964	95.75243	
40	M40: f = 980.91	95.86084	93.71964	95.78288	
41	M41: f = 982.93	95.86084	93.72488	95.78288	
42	M42: f = 1,020.4	95.86084	93.73917	95.78288	
43	M43: f = 1,021.4	96.38815	93.73917	95.86406	
44	M44: f = 1,062.0	96.38815	94.16627	95.86406	
45	M45: f = 1,071.5	96.42472	94.16627	96.34676	

JOB NO.		SHEET	9 OF 9
PHASE	Final Design	TASK	Wall 09.05R-B
JOB NAME	I-405 Design Build		
BY	DMP	DATE	9/21/2020
CHECKED BY	EHP	DATE	9/22/2020

Colorado Center Tower II
2000 S. Colorado Blvd., Ste 2-1000
Denver, CO 80222
+1 (303) 935-6505 Fax '+1 (303) 935-6575

Results



100% Direction 1 - 30% Direction 2

Joint	Result Case	Force X (kip	Force Y (kips)	Force Z (kips)	Moment X (kips-f	Moment Z (kips-f
24	100% Directi	0.23	0.64	4.21	1.51	0.02
73	100% Directi	0.51	0.51	3.19	1.28	0.05
121	100% Directi	0.56	0.97	2.36	0.77	0.20
170	100% Directi	0.34	0.72	1.01	0.39	0.30
220	100% Directi	0.83	1.70	0.87	0.53	0.68
270	100% Directi	2.77	6.47	1.85	4.42	1.20
313	100% Directi	2.12	0.97	0.53	2.70	0.18
351	100% Directi	1.89	0.23	0.47	2.51	0.03
389	100% Directi	1.74	0.29	1.18	2.35	0.02
427	100% Directi	1.60	0.35	2.05	2.22	0.02
465	100% Directi	1.45	0.62	3.26	2.10	0.05
503	100% Directi	0.70	0.66	5.18	1.69	0.20
			14.14		22.48	

Delta To, k = 15.71
M k-ft / ft = 6.24
V k / ft = 3.93

30% Direction 1 - 100% Direction 2

Joint	Result Case	Force X (kip	Force Y (kips)	Force Z (kips)	Moment X (kips-f	Moment Z (kips-f
24	30% Directio	0.07	2.14	1.26	5.05	0.05
73	30% Directio	0.15	1.69	0.96	4.26	0.17
121	30% Directio	0.17	3.25	0.71	2.57	0.67
170	30% Directio	0.10	2.39	0.30	1.31	1.01
220	30% Directio	0.25	5.67	0.26	1.77	2.27
270	30% Directio	0.83	21.56	0.55	14.73	3.99
313	30% Directio	0.64	3.24	0.16	8.99	0.58
351	30% Directio	0.57	0.77	0.14	8.37	0.09
389	30% Directio	0.52	0.97	0.35	7.84	0.07
427	30% Directio	0.48	1.16	0.62	7.41	0.06
465	30% Directio	0.44	2.08	0.98	7.00	0.16
503	30% Directio	0.21	2.22	1.55	5.63	0.65
			47.12		74.92	

JOB NO.	PS19203160	SHEET 1 OF 4	
PHASE	Design	TASK	Wall Cap Connection
JOB NAME	I-405; Renton To Belleview Widening and Express Toll Lanes Project		
BY	JDF	DATE	9/28/2020
CHECKED BY	DGB	DATE	9/30/2020

wood.
Colorado Center Tower II
2000 S. Colorado Blvd., Ste 2-1000
Denver, CO 80222
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Concrete Cap Connection Check - FALL PROTECTION ONLY

Input

Analysis H = 4.33 ft — Rail h = 3.5' + cap thickness to top of wall = 0.83' (from load at top of rail to fixed based in concrete fascia)

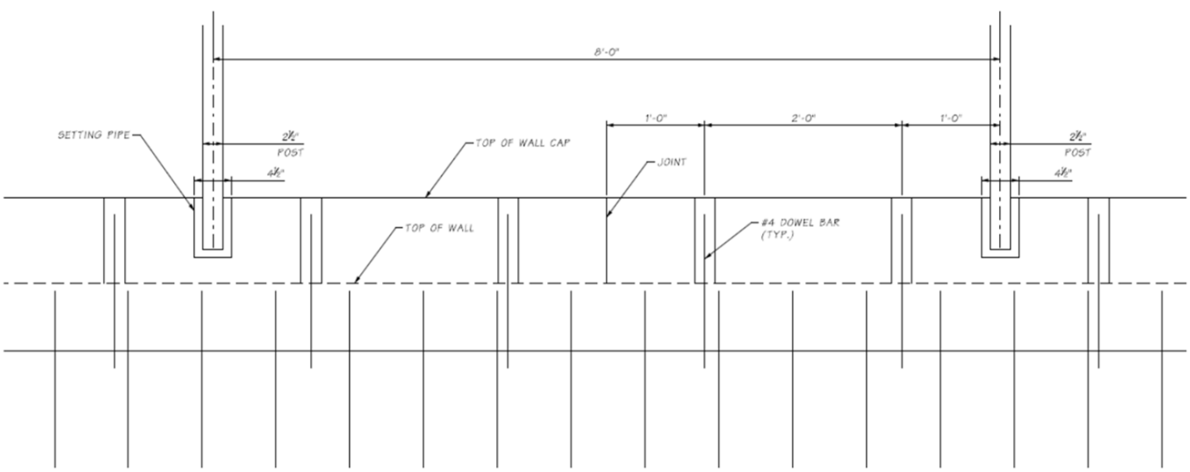
stem width = 1.25 ft — (at top of panel - total thickness)

Load Factors

design check with Strength I limit state: PL 1.75 — LRFD T 3.4.1-1

Fall Protection Load

Loading would transfer into the wall via the wall connection/rebar spaced at an interval to resist the loading between the 8-ft post locations. Assume precast coping to be provided at no more than 8-ft sections per discussion with contractor. Therefore 1 post would act on one coping section. Assume 1 bar on each side of post resists post loading.



P_{LL} = 0.20 k (unfactored) (Per Post)
0.35 k (factored)

Materials Input

			Reference
f'c =	4.0	ksi	BDM 5.1.1.B.2
β1 =	0.85		And per Plan Sheets
fy =	60.0	ksi	BDM 5.1.2
γ_e =	0.75		LRFD 5.6.7
modulus of rupture coefficient =	0.24		LRFD 5.4.2.6
fr =	0.480	ksi	
wc =	0.155	kcf	BDM 3.8
Ec =	4576	ksi	LRFD 5.4.2.4-1
n = 29000 / 4576 =	6.34		
λ =	1.0		LRFD 5.4.2.8

$$0.24\,\lambda\sqrt{f'_c}$$

$$E_c = 120{,}000 K_1 w_c^{2.0} f'^{0.33}_c$$

JOB NO.	PS19203160	SHEET	2 OF 4	
PHASE	Design	TASK	Wall Cap Connection	
JOB NAME	I-405; Renton To Belleview Widening and Express Toll Lanes Project			Colorado Center Tower II
BY	JDF	DATE	9/28/2020	2000 S. Colorado Blvd., Ste 2-1000
CHECKED BY	DGB	DATE	9/30/2020	Denver, CO 80222
				+1 (303) 935-6505 Fax '+1 (303) 935-6575

Structural Design Loads

Stem Loads

Analysis H = 4.33 ft (height above top of stem where load acts)

Shear

Vu = 0.35 k

Moment

Mu = 1.52 k-ft (Analysis Ht * Vu)
Ms = 0.87 k-ft (as above, unfactored)

Minimum Reinforcement

Minimum Reinforcement LRFD 5.6.3.3
1.33 Mu = 2.02 k-ft

Stem Loading Summary

Vu = 0.35 k
Mu = 2.02 k-ft
Mserv = 0.87 k-ft

Flexural Design

Φ Flexure = 0.9 assumes tension controlled LRFD 5.5.4.2

Stem

b = 48.00 in Tributary Width of Coping Containing Two Connection Bars (1 Ea. Side of Post)
h = 15.00 in
distance to bars = 7.50 in (at mid of panel section)
Bar size = 4
0.20 sq in / bar

Coping Embed = 8.00 in Depth Connection is Embedded into Precast Coping
Development Length = 14.40 in Development Length of Connection Bar

$$de = 15.00 - 7.50 - \frac{4}{16} = 7.25 \text{ in}$$

$$dc = 7.50 - \frac{4}{16} = 7.75 \text{ in}$$

\# Bars (Legs) = 2
As = 0.222 sq in Efffective Area (Based on Partial Development)

$$\text{stress block depth, } a = \frac{As \times fy}{0.85 \times f'c \times b} = 0.08 \text{ in}$$

Φ Mn = 0.90 x As x fy (de - a/2) = 7.21 k-ft > 2.02 OK

JOB NO.	PS19203160	SHEET	3	OF	4
PHASE	Design	TASK	Wall Cap Connection		
JOB NAME	I-405; Renton To Belleview Widening and Express Toll Lanes Project				
BY	JDF	DATE	9/28/2020		
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wood.
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Crack Control

LRFD 5.6.7

Crack control is required where the tension in the section exceeds 80% modulus of rupture (fr) = 0.480 ksi

80%fr = 0.384 ksi

Compute section area moment to equal criteria above and compare to moment from loading:

$$S = I/c = \frac{=1/12\,(b)\,(h)^3 =}{c} \quad \frac{13500}{7.5} = 1800 \text{ in}^3$$

M*c/I = 0.384
M = 57.6 k-ft **> 0.87 OK** **Crack control Not Required**

reinforcement ratio $\rho = \frac{As =}{b\ de}$ $\frac{0.222222222}{12.00 \times 7.25} =$ 0.0026

$$k = \sqrt{2n\rho + (n\rho)^2} - n\rho = \quad 0.164$$

$$j = 1 - k/3 = \quad 0.95$$

fss = Min (0.6 Fy, $\frac{Mserv}{As\ (j)\ de}$) = 6.83 ksi

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} = 2.53$$ LRFD 5.6.7-2

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c = 14.92 \text{ in}$$ LRFD 5.6.7-1

> 2 OK

Shear Design

LRFD 5.7.3.3

Φ Shear = 0.90
β = 2.0
θ = 45° LRFD 5.5.4.2
bv = 12 in LRFD 5.7.3.4.1

Stem

0.9 de = 0.90 x 7.25 = 6.53 in
0.72 h = 0.72 x 15.00 = 10.80 in LRFD 5.7.2.8
dv = max(0.9 de, 0.72 h) = 10.80 in

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}\ b_v d_v$$ LRFD 5.7.3.3-3

Vc = 16.38 k
Φ Vn = 14.74 k
> 0.35 OK

JOB NO.	PS19203160	SHEET 4 OF 4		
PHASE	Design	TASK	Wall Cap Connection	
JOB NAME	I-405; Renton To Belleview Widening and Express Toll Lanes Project			Colorado Center Tower II
BY	JDF	DATE	9/28/2020	2000 S. Colorado Blvd., Ste 2-1000
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wood.

Shear Friction Interface Check

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c)$$

LRFD 5.7.4.3-4

Ignore c cohesion)

Ignore Pc - net compressive load)

Φ Vn = 0.9*µ*Avf*fy

LRFD 5.7.4.4

Avf = 0.2222 in^2 (#4 bar)

fy = 60.0 ksi

µ = 0.6

Φ Vn = 7.20 k

> 0.35 OK

Cohesion and Friction Factors

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened:

c = 0.075 ksi
μ = 0.6
K_1 = 0.2
K_2 = 0.8 ksi